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This report presents the description and preliminary results through the first year of the work conducted under the research project entitled "Evaluation of Prestress Loss Characteristics of In-Service Bridge Beams", (PennDOT Research Project 71-9, Lehigh University Project No. 382).

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RESEARCH PROJECT 71-9

Evaluation of Prestress Loss Characteristics of
In-Service Bridge Beams

SOME OBSERVATIONS ON THE PRESTRESS LOSS BEHAVIOR OF BEAMS IN AN EXPERIMENTAL BRIDGE

by

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Bethlehem, Pennsylvania

July 1975

Fritz Engineering Laboratory Report No. 382.2

ABSTRACT

This report presents the description and preliminary results through the first year of the work conducted under the research project entitled "Evaluation of Prestress Loss Characteristics of In-Service Bridge Beams", (PennDOT Research Project 71-9, Lehigh University Project No. 382).

By strain measurements on field stored specimens as well as control specimens stored in the laboratory, it was found that indoor specimens suffer somewhat higher shrinkage and creep strains than their outdoor counterpart. Little difference was observed between the members containing stabilized strands and those containing stress-relieved strands. The transfer length for 1/2" strands was observed to be approximately 30 in. Also, preliminary examination indicated that the thermal prestress loss due to elevated curing temperature was completely recovered after the end of curing.

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1. INTRODUCTION

1.1 Background

In the design of prestressed concrete members, the estimation of prestress losses is of the utmost importance. When prestressed concrete was first being used, the loss estimation was done by allowing a fixed value or a fixed percentage of initial stress. While the various sources for the losses were recognized, insufficient knowledge prevented accurate estimation of the components either individually or collectively. Later, numerous research works have led to the development of more sophisticated formulas, taking into consideration the influence of many important parameters, and also reflecting the interaction of the several time-dependent components. However, most of this research work, whether directly on the problem of prestress losses, or on the basic properties of concrete, were performed using specimens fabricated and stored in laboratories. Only a few studies have been made on actual in-service structures. Recently, some indication has been found that members in a varying environmental condition may not behave in the same manner as those under a constant environment. Thus, a fundamental question was raised as to the applicability of the laboratory-based formulas to actual design use.

For several years since 1966, an extensive research project has been conducted at Lehigh University aimed at establishing a rational method for the estimation of prestress losses in pretensioned bridge members (PennDOT Research Project 66-17, Lehigh University Project 339).

Concrete specimens with and without prestress were measured for their instantaneous and time-dependent strains. In addition, strand specimens were tested to determine their relaxation characteristics. Regression methods were used to develop estimation formulas for the elastic, shrinkage, creep and relaxation losses of prestress. These formulas were then combined and a rational analytical method was established for the estimation of prestress losses. Inasmuch as all specimens used in that project were stored in Fritz Engineering Laboratory under reasonably stable environmental conditions, the conclusions were subject to the aforementioned fundamental question. The research described in this report was conducted primarily in order to provide an answer.

1.2 Problem Statement and Objectives

The research project reported herein, entitled "Evaluation of Prestress Loss Characteristics of In-Service Bridge Beams" (PennDOT Research Project 71-9, Lehigh FL 382), was initiated in the fall of 1971 in response to a concern that the prestress loss estimation procedures generated in the preceding project (PennDOT 66-17, Lehigh 339) may not be directly applicable to bridge members under actual service condition. As mentioned in Section 1.1, there has been some indication that a varying environmental condition may cause the prestressed concrete members to show a prestress loss different from its counterpart stored under a uniform average environment. The study included a field investigation of several in-service bridge beams and a comparison with predictions based on the previous research.

The objectives of this research project are as follows:

1. To determine the prestress loss in pretensioned bridge members under an outdoor in-service condition.

2. To establish a relationship between the prestress loss behavior of laboratory specimens and in-service members.

3. To test the prediction formulas proposed by Project 66-17, and to adjust the same, if necessary.

4. To identify areas where additional research may be needed.

5. To provide an in-service comparison of the prestress loss characteristics of pretensioned bridge members containing the low-relaxation strands with those containing the stress-relieved strands.

6. To verify the transfer length of 1/2 in. diameter prestressing strands.

7. To identify the influence of strand diameter on the prestress loss of the member.

8. To identify the effect of differential shrinkage of the deck and beam concretes on the prestress loss of the beam.

1.3 Description of the Experimental Bridge

The field study of the project is being carried out on an experimental bridge which is part of a pavement durability test track located near State College, Pennsylvania. Two other research projects under the auspices of the Pennsylvania Transportation and Traffic Safety Center (PTTSC) of the Pennsylvania State University are simultaneously being conducted at the same site. The activities of the three concurrent projects are coordinated by the Pennsylvania Department of Transportation, Bureau of Materials, Testing and Research. Brief descriptions of the PTTSC projects are given in Section 1.4.

The experimental bridge is a two-span structure carrying the test track over an access road. It is located on a 1% grade and a curve of 550 ft. radius, with a superelevation of 0.1040 ft. per ft. The access road had a grade of 6%. The superstructure spans are 60 ft. center to center of bearings. The deck structure is approximately 36 ft. wide between safety curbs, and its thickness is an equivalent of 7-1/2 in. Six precast pretensioned concrete I-beams are used in each span, at spacings of 6 ft. 10 in., as the main structural members. Figures 1 and 2 show a plan view and a cross section of the bridge.

The deck structures are varied for the two spans. For one span, the conventional cast-in-place concrete slabs are used. For the second span, 3 in. thick precast pretensioned planks are used, combined with a 4-1/2 in. cast-in-place concrete topping.

Many design decisions on this experimental bridge were made in consideration of the two PTTSC research projects. A complete description of the bridge structure, its fabrication and erection, and the background of many design decisions are reported by the Pennsylvania State University[16]. In Chapters 2 and 3 of this report are details in design, fabrication and erection of the experiment bridge which are relevent to the prestress loss study.

1.4 Concurrent PTTSC Research Projects

As indicated earlier, two PTTSC research projects are being concurrently conducted at the experimental bridge site. Here are given very brief descriptions of these two projects. For further details the readers are referred to documents by the researchers thereof.

PennDOT Research Project 71-7, entitled "An Evaluation of Pennsylvania's Flexible Pavement Design Methodology" is aimed at developing engineering facts and criteria which can be used for the design, construction, maintenance and improvement of flexible pavements. The Pavement Durability Test Track was designed and constructed specifically to serve the purpose of this investigation. This test track is oval shaped and approximately one mile long. Varied pavement designs (surface, base course and subbase materials and thicknesses) are used at different segments of the track in order to compare their performance under load. A specially designed vehicle with five axles carrying variable loads is used to simulate the traffic loads. During the first cycle of this study, covering a period of three years, a mixed loading spectrum representing the equivalent of over one million applications of an 18-kip axle on the pavement is planned.

The basic purpose of PennDOT Research Project 71-8, entitled "An Experimental Prestressed Concrete Bridge" was to evaluate the behavior of various types of bridge deck structure under traffic loading and deicing agents. Two basic types of deck structure, as described earlier in Section 1.3, are included. Two variations were used for each type of structure. For the cast-in-place concrete slab, both removable wood form and left-in-place metal deck form were used, each for half of the span. For the precast prestressed planks, two kinds of joints, butt and beveled, were used. The traffic load on the bridge is identical to that used in Project 71-7 for the test track. During the winter months, after each accumulation of snow or ice, two deicing agents are applied to selected sections of the bridge deck for observation of their effects. This project also includes a study of the structural behavior of the main beams throughout the life of this experimental bridge. Camber of beams is being recorded from the fabrication time, through transportation, erection and the application of traffic loads. In addition, during specified breaks of the simulated traffic run, static tests using a vehicle representing HS20-44 standard loading at crawl speed were conducted. After the completion of traffic load test, it is planned to test the superstructure with progressively increasing live loads, until the structure is judged to be no longer serviceable.

2. DESIGN OF SPECIMENS

2.1 Materials

The materials used for all specimens of this project were required to satisfy the quality inspection specifications of the Pennsylvania Department of Transportation. The handling of materials, and the placing and curing of concrete were subjected to the standard inspection procedures of that department.

The concrete for the pretensioned beam members was specified to have a minimum compressive strength of 5000 psi at transfer of prestress and 5500 psi at 28 days. The concrete for cast-in-place deck slab is specified to be Class AA, with a 28 day strength of 3750 psi. The prestressing strands were 1/2 in. diameter seven-wire strands of the 270 K grade, with a minimum specified tensile strength of 41300 lbs. per strand. Eight of the twelve main beams were prestressed with the conventional stress-relieved strands, while the new low-relaxation stabilized strands were used in the other four beams. All prestressing strands were supplied by the CF & I Steel Corporation, Roebling Division.

The basic mechanical properties of the materials actually used were determined from standard concrete cylinders molded during fabrication time, and from strand specimens taken from the same reels.

2.2 Main Beam Members

The structural design of the experimental bridge was done by the staff of the Pennsylvania Department of Transportation, Bridge Division, based on a service live load of the HS20-44 class and in accordance with the standard design procedures of that office. For the main superstructure beam members, the PennDOT standard 20/33 pretensioned concrete I-beams were selected. This section is considerably shallower than what would normally be used for the particular span and live load, but was chosen in order to accentuate the flexural behavior of the superstructure under live load and to exagerate the live load effects on the deck structure, in consideration of the concurrent PTTSC Project PennDOT 71-8.

The design of the main beam members required an initial prestress of 983 kips, at an eccentricity of 7.95 in. at midspan and 2.04 in. at the ends. This prestress was supplied by thirty-four 1/2 in. strands, of which fifteen were harped at two points, 10 ft. each side of the midspan section. Figure 3 shows the cross section of the main beams, the profile of prestress and the arrangement of strands.

Six of the twelve main beams were instrumented for concrete strain measurements. These include all four containing the stabilized strands and two containing the conventional stress-relieved strands. The instrumentation consisted of Whittemore gage targets at 10 in. gage lengths near the midspan section on both sides of the beam. The construction of these target points was identical to those used in the previous project (PennDOT 66-16), and has been fully described elsewhere[2,6]. As shown in Fig. 4, targets were installed at four levels across the depth of the beam, so that the full distribution of concrete strains can be determined.

2.3 Supporting Specimens

Eight short specimens were fabricated together with the main beams for control measurements of shrinkage and prestress strains. They all have the same cross section as the main beams. Four of these are six foot long, and contain thirty-four pretensioned strands. As no prestress was introduced into these specimens, they were subjected to shrinkage strain only. The other four short specimens are seven feet long and were subjected to the same prestress as the main beams, but at a constant eccentricity of 5.15 in. This reduced eccentricity was selected so that the stress condition in these short prestressed specimens would be the same as that at the midspan section of the main beams under full design dead load. It was reasoned that by simulating the stress condition this way, the creep strains would also be comparable. Two of these short prestressed specimens contain stabilized strands, and the others contain stress-relieved strands.

The shrinkage and short prestressed specimens were instrumented as shown in Fig. 4. Whittemore gage target points were installed at four levels at the mid-section of each member on both sides. In addition, a string of target points, at 5 in. intervals were installed on the short prestressed specimen along the c.g.s. line. Measurements along this line were used for the determination of transfer length of the prestressing strands.

2.4 Arrangement of Specimens

Two short prestressed specimens, one containing stress-relieved strands and the other stabilized strands, are stored in the Fritz Engineering Laboratory of Lehigh University. Three shrinkage specimens were used as supports in mechanical tests on the precast planks carried out under the PTTSC Project PennDOT 71-8, and are stored in the Civil Engineering Laboratory of the Pennsylvania State University. These five specimens are being subjected to essentially constant (and rather dry) environments. The other three short specimens are stored near the main beams at all times and are therefore subjected to a varying and considerably wetter environment. At the site of the experimental bridge, these short specimens are placed underneath the bridge near the abutments, hence not directly exposed to sunshine or precipitation.

The instrumented main beams are located at the third, fourth and the fifth positions, from the outer side of the test track, of both spans, as indicated in Fig. 2. The two beams containing stress-relieved strands occupy the third position in one span and the fifth in the other. This crisscross arrangement was intended to facilitate a separation of the effects of several factors: the relaxation characteristics of the strands, the live load stresses, and the interaction between the beam and the deck structure. Fig. 1b shows the framing plan of the main beams.

Table 1 lists all concrete specimens used in the project for long term strain measurements and shows the particulars of each specimen.

3. FABRICATION AND ERECTION

3.1 Fabrication

The fabrication of the superstructure beams and the short specimens was done by Schuylkill Products, Inc. at Cressona, Pennsylvania, on contract basis. The contract was administered through the Pennsylvania State University under a companion project (PennDOT Research Project 71-8). The specimens were cast in three fabrication runs over a period of two weeks from January 26 to February 9, 1972. In the first run, four main beams containing the stress-relieved strands were fabricated. In the second run, four main beams, two 7 ft. creep specimens and two 6 ft. shrinkage specimens, all containing stress relieved strands, were fabricated. In the third run were fabricated the remaining four main specimens, two 7 ft. creep specimens, and two 6 ft. shrinkage specimens, all with stabilized strands. All instrumented specimens were fabricated in the second and third runs. A large number of standard cylinders were also taken from each run to be tested at various times for quality control.

The first step in each fabrication run was the feeding of the strands through the bulkheads. Load cells were placed on four selected strands (two straight and two draped) at the dead end for the purpose of monitoring the strand force. The locations of the load cells are shown in Figs. 3a and 3b. A number of deflecting devices were placed at appropriate locations, and the "draped" strands were passed through these devices to achieve the desired variation of eccentricity. Each strand was first individually tensioned up to 3000 lbs. (approximately 20 ksi) to take up slack and to detect and remove any possible tangling. The tensioning at this step was done from the "dead end" of the prestressing bed. By the normal procedure used by this fabricator, group tensioning would next be used to achieve the full initial prestressing stress of 189 ksi (28.9 kips per strand). However, during the first fabrication run for this experimental bridge, two attempts of group tensioning both resulted in wire or strand breakage, and the entire process had to be restarted. It was felt that the difficulty probably lay in the large number of deflecting points used (for the four beams in this run, 8 hold-down and 5 hold-up devices were needed) and the excessive frictional resistance caused by these devices. In order to alleviate this situation, strands were individually tensioned to the full value at this second stage on the third attempt, and no breakage occurred. The individual tensioning process was then used for the second and third fabrication runs. Considerable time was needed to adjust the hold-down devices after tensioning, the total time from initial stretching to completion of tensioning was nearly six hours in each case. Figure 3b shows the typical bed layout during fabrication, and the specimens cast during fabrication runs 2 and 3, respectively.

After the strands were tensioned, the non-prestressed reinforcement was tied in, the side forms for the concrete beams were put in place, and the casting of the concrete was begun. Eight batches of concrete were used in the first fabrication run, and nine batches were used for each of the two subsequent runs. After placing the concrete, steam started curing the beams at approximately 140° F. The average length of curing approximately twenty hours. At the end of this period, one cylinder from each batch of concrete was tested for compressive strength to assure the achieving of the required release strength before curing was terminated.

For the transfer of prestress, the top nine draped strands were first flame-cut between each pair of specimens before releasing the strand-deflecting devices. As the hold-down forces were calculated to be greater than the weight of the concrete member, this special procedure was needed in order to assume that the beams would not be lifted off the bed. After the removal of side forms and releasing of deflecting devices, group detensioning was then applied to the rest of the strands. Finally, strands between beams were flame-cut to the end surface of the members.

Load-cell readings were taken throughout the fabrication period, from initial threading until the completion of detensioning. Readings were taken at several stages of tensioning, and detensioning, and several times during the curing period. The final load-cell readings were taken after the strands had been detached from the bulkhead, and the load-cells became completely unloaded. This set of readings was taken to ascertain the zero drift of these devices.

For the second and third runs, when the instrumented specimens were fabricated, target points for concrete strain measurements were installed. Each target point for strain measurement consisted of a brass insert and a stainless steel contact seat. Before the assembling of the beam forms, the brass inserts coated with a layer of fine sand were attached to the inside of the side forms at predetermined positions. Details of the target points are found in several previous reports from

Lehigh University[2,6]. At the end of curing before detensioning, the side forms were removed and stainless steel contact seats were screwed tight into the brass inserts. A set of Whittemore gage readings were immediately taken. This set of readings, taken before transfer and corresponding to zero concrete strains, was used as the basis of reference in the future. After transfer, when the specimens had been separated from each other and from the abutment, one end of each specimen was lifted slightly to relieve any friction between the specimen and the bed. The "after transfer" concrete strain readings were then taken. Concrete strains were not measured after the cutting of the top strands, since that represents a partial transfer and an atypical condition.

Throughout the curing period, concrete temperature was automatically recorded at three stations along the prestressing bed.

The total time of fabrication, from initial threading of strands to the completion of detensioning, was approximately 70 hours. Table 2 shows the sequence of activities for runs 2 and 3, when specimens used in this project were fabricated.

3.2 Transportation and Loading

The beam members were temporarily stored in the yard of the fabricator while the earthwork and substructures were being built at the bridge site. On May 22, 1972, the beams were transported to the test track site and placed on the abutments and pier immediately. At this time, the age of the instrumented beams was 109 days for those containing stress-relieved strands and 104 days for those containing stabilized strands. The construction of the deck structure soon followed, and was completed by July 24, 1972 (age 172 and 167 days). Experimental traffic started to move over the test bridge during the last part of September, 1972. The test track was formally opened on October 3, 1972 (approximately 240 days). The specially designed test vehicle started operation later during the same month.

The prestressed short specimens P2 and P3 were moved to Fritz Laboratory for storage and observation on February 15, 1972. Three shrinkage specimens, S1, S2 and S3 were moved indoors on March 15, 1972, to the Civil Engineering Laboratory of the Pennsylvania State University. The other short specimens, P1, P4 and S4 were stored with the main beams and were transported together with the beam members to the test bridge site. They were placed under the bridge in front of the abutments.

4. TESTS AND RESULTS

4.1 Basic Properties of Concrete

Numerous concrete cylinders were tested to establish the fundamental properties of concrete. Compressive strength was determined at the fabricating plant immediately before transfer of prestress. Additional cylinders were tested at Fritz Engineering Laboratory for compressive strength and modulus of elasticity soon after transfer, at 28 days and at several subsequent times relevant to the changing of loading conditions in the bridge members.

The tests were carried out in accordance with ASTM specifications, C39 and C469, using a 300,000 lb. hydraulic universal testing machine, a mechanical compressometer with a 12" equivalent gage length, and a 0.0001 in. dial gage. The modulus of elasticity was based on a straight line between a point corresponding to 50 μ in./in. of strain and a point at 40% of ultimate load, in accordance with ASTM standard C469. The average values of concrete strength and modulus of elasticy at various times are listed in Table 3.

4.2 Basic Properties of Strands

Three reels of the conventional stress-relieved strands and two reels of the stabilized strands were used in fabricating the beam members and short supporting specimens. Three 7-foot specimens were taken from each reel and tested for their diameter, wire diameter, lay, and the mechanical properties. The results of these tests, performed at the Fritz Engineering Laboratory, are shown in Table 4, together with results provided by the supplier as well as obtained by the laboratory of PennDOT, Bureau of Materials, Testing and Research.

The tensile tests of strands were made on a 60,000 lb. Tinius-Olsen hydraulic universal testing machine. The wires of the strand were welded together at each end to ensure uniform elongation of all wires. To prevent premature failure of wire inside of the strand chucks, two brass bars were used within the grips of the machine heads in front of the strand chucks, so that the chucks would not be required to transfer the full strand force. Strain measurement was made by a clamp-on device using two dial gages and a gage length of 24 in. Both the brass bar grips and the strain measuring setup have previously been used in other Lehigh projects (339, 309) and detailed descriptions are found elsewhere[1,7]. After the measured stress-strain values have been plotted, the modulus of elasticity was calculated as the slope of the line connecting the two points on the curve, corresponding to stresses of 26.2 ksi and 160 ksi, respectively. The yield strength was determined as the stress value corresponding to a total strain of 0.01, as defined by ASTM standard A416.

4.3 Strand Forces Prior to Transfer

During the fabrication of the specimens, four load cells were used at the dead end of the prestressing bed to monitor the changes in strand forces. Two of these were placed on straight tendons and the other two were on draped ones (Fig. 3). These load cells were specially designed for strand force measurement in order to get a high degree of precision as well as long-term stability. Details of the load cells are described in Fritz Engineering Laboratory Report No. 339.5 (Ref. 7). The load cell readings were taken at various times, starting from the initial tensioning until the completion of detensioning of the strands. The strand forces measured this way were assumed to represent the average force in straight and draped strands, respectively. The variation of these average strand forces are presented graphically in Figs. 5 and 6, together with the variation of concrete temperature during the curing period.

4.4 End Development of Prestress

The prestress development in the end regions of pretensioned members was studied by means of measuring concrete strains in the short prestressed specimens. As shown in Fig. 4, Whittemore gage target points were installed along the c.g.s. line of these specimens at 5 in. spacings. Readings were taken for overlapping 10 in. gage distances, first, immediately before the flame-cutting of the nine top strands, and again immediately after the completion of transfer. On account of the rather complicated detensioning procedure, approximately four hours of time elapsed between these two sets of readings. Considerable shrinkage and thermal contraction took place during this period, and these strains must be deducted from the measured total change of gage distance to obtain the elastic strain due to prestress. For this purpose, the shrinkage and contraction in these short prestressed specimens were assumed to be uniform over their length, and equal to the average strain measured in the companion shrinkage specimens. Thus, the elastic concrete strains in the short prestressed specimens were obtained by subtracting from the measured total strains a uniform value equal to the average strain observed in the companion shrinkage specimens.

An examination of the concrete elastic strain data over the length of the short prestressed specimens quickly revealed that it was nearly constant over the middle portion, and decreases smoothly towards both ends. This observed distribution indicates that full prestress was attained near the middle of the member, where steel and concrete deformed compatibly. Outside of this middle region, prestressing steel slipped inward and stress developed gradually.

A more precise study of the prestress development was carried out by actually converting the elastic strain values into steel stress values. For the middle portion, this conversion can be easily accomplished by imposing the strain compatibility relationship:

$$\epsilon_{s2} - \epsilon_{so} = \epsilon_{co}$$

where

ϵ_{s2} = steel strain before transfer

ϵ_{so} = steel strain immediately after transfer

ϵ_{co} = concrete elastic strain at c.g.s., immediately after transfer.

Multiply the preceding equation with E_s and rearrange, the steel prestress immediately after transfer is obtained.

$$f_{so} = f_{s2} - E_s \epsilon_{co} \tag{4-1}$$

where

f_{so} = steel prestress immediately after transfer, in ksi

f_{s2} = steel stress before transfer, in ksi

E_s = modulus of elasticity of prestressing steel, in ksi

Outside of the middle region, strain compatibility does not hold and Eq. (4-1) is not applicable. The steel prestress was calculated indirectly by the equilibrium condition. For any cross section, whether inside or outside of the transfer regions, the equilibrium of internal stresses requires[14]:

$$f_{cs} = A_{ps}\, f_{so} \left(\frac{1}{A_n} + \frac{e_n^2}{I_n}\right) - M_g \left(\frac{e_n}{I_n}\right) \tag{4-2}$$

where

f_{cs} = concrete fiber stress at c.g.s., immediately after transfer

= $E_{ci}\, \epsilon_{co}$, in ksi

A_{ps} = area of prestressing steel, in sq. in.

A_n = area of net concrete cross section, in sq. in.

e_n = eccentricity of prestress, referring to net concrete section, in in.

I_n = moment of inertia of the net concrete section, in in.^{4}

M_g = bending moment caused by member weight, in kip-in.

E_{ci} = modulus of elasticity of concrete at transfer time, in ksi

For the short prestressed specimens, M_g is extremely small and the second term in Eq. (4-2) may be neglected. Also, in a previous research report[15], it has been established that the first term in Eq. (4-2) could be evaluated using gross section properties by introducing a dimensionless geometrical parameter β:

$$\beta = \frac{1}{A_{ps}\left(\frac{1}{A_g} + \frac{e_g^2}{I_g}\right)}$$

where

A_g, e_g, I_g = area, eccentricity and moment of inertia, respectively, of the gross cross section, all in inch units

Eq. (4-2) is now transformed into the following form:

$$f_{cs} = f_{so}/(\beta - 1) \qquad (4\text{-}2a)$$

The equivalence of Eq. (4-2a) to Eq. (4-2) has been given in reference 15. Rearranging terms in Eq. (4-2a),

$$f_{so} = E_{ci}\ \epsilon_{co}\ (\beta - 1) \qquad (4\text{-}3)$$

Eq. (4-3) is suitable for estimating f_{so} from the observed ϵ_{co} values. In order to assure continuity of the stress development curve, it was noted that both Eqs. (4-1) and (4-3) must be satisfied in the middle region of the specimen, where both equilibrium and compatibility prevail. Combining these two equations

$$f_{s2} - E_s\ \epsilon_{co} = E_{ci}\ \epsilon_{co}\ (\beta - 1)$$

Therefore

$$E_{ci} = \frac{f_{s2} - E_s\ \epsilon_{co}}{(\beta - 1)\ \epsilon_{co}} \qquad (4\text{-}4)$$

For the construction of Figs. 7 and 8, the initial concrete modulus E_{ci} was calculated from Eq. (4-4), using the average concrete elastic strain for the middle 20 inches for ε_{co}. Equation (4-3) was then used to calculate f_{so} values outside of the middle segment. The stress values for the middle segment were calculated from Eq. (4-1), and did not depend upon the concrete modulus.

4.5 Concrete Strains

Whittemore gage strain readings were taken from the beam members as well as the supporting specimens before and after transfer, and at preselected gradually increasing time intervals as listed in Table 5. All gage distances were 10 in. and the instrument used had a finest division of 0.0001 in. The difference between the before and after transfer readings provided information on the elastic concrete strains. Long-term concrete strains were calculated as changes from the after transfer readings.

All strain measurements used for the elastic and long-term strain investigations were taken near the middle section of the specimens. The long string of strain targets along the c.g.s. line of the short prestressed specimens were used for transfer length study only, and were removed after the "after - transfer" readings had been completed. As shown in Fig. 4, strain measurements were made at four levels in each specimen, so that the complete strain distribution can be determined. In the beam members and the short prestressed specimens, four strain measurements were made at each level (two on each side). Their averages were used as the experimental strain values for the level. For the shrinkage specimens, there was only one gage distance on each side at each level. However, as these specimens were completely free from loads and stresses, it was reasonable to assume a uniform strain distribution. Therefore, strain values from all four levels were averaged into one single value for the purpose of comparison.

Figures 9 and 10 show the distribution of measured concrete strains in specimens immediately after transfer. Figures 11 and 12 show the distribution of long-term strains for two beam members at various times. In all four figures, the essentially linear strain distribution is clearly seen. Figure 13 shows the time variation of the average shrinkage strains and Fig. 14 shows the variation of the long-term concrete strain at the c.g.s. line in the short prestressed specimens. Similar graphs have been plotted for other specimens and locations, but are not included in this report since they merely repeat the characteristics of these typical diagrams.

5. DISCUSSION

5.1 Material Properties

5.1.1 Properties of Concrete

In Table 3 are listed the design (desired) values and the experimental (actual) values of the several concrete material properties. Each test value represents the average of two to three standard cylinder tests. The concrete cylinder strength was within 2 to 3% of the design value at the transfer time, but was significantly higher at later ages. For the second and third runs of fabrication (all instrumented specimens were fabricated in these runs), the concrete strengths determined immediately before transfer were 5.11 ksi and 4.96 ksi, respectively, while the required value was 5 ksi. At 28 days, the experimental values were 7.4 ksi and 7.74 ksi, respectively, for these two runs; while the design value was 5.5 ksi. Clearly, in order to satisfy the required release strength, the concrete mix chosen was inherently much stronger than that required for the specified 28 day strength.

It was noted that the experimental values of modulus of elasticity deviated some from the standard formula recommended by ACI.

$$E_c = 33\, W^{1.5}\sqrt{f_c'}$$

where

E_c = modulus of elasticity of concrete, psi

w = weight of concrete, lb. per cu. ft. (taken as 149 lb./cu.ft.)

f_c' = specified compressive strength of concrete, psi

A direct comparison indicated the above formula to be approximately 10% too high. The ACI formula was based on an empirical study of initial tangent or secant modulus while the testing in this study used a "chord" method as specified by the ASTM Standard Method, thus accounting for the difference. On several occasions, the strain measuring device was either unavailable or not functioning properly. For these occasions, the modulus values listed in Table 3 were estimated as 90% of the calculated value by the ACI formula.

5.1.2 Properties of Prestressing Strands

The properties of the strands used in this project, as determined by the project staff, are listed in Table 4, together with similar information supplied by the material supplier (CF & I Steel Corporation, Roebling Division) and the Pennsylvania Department of Transportation, Bureau of Materials, Testing and Research.

It is noted that the stabilized and stress-relieved strands had nearly identical physical properties, with one possible exception. The yield strength of the stabilized strands appears to be significantly higher than that of the stress-relieved strands. This difference is believed to be a result of the special "stabilizing" treatment given the stabilized strands. It is also noted that no systematic discrepancy exists among the mechanical properties from the three sources. The largest devition is less than 5%, while in several cases, data from all three sources agree within 1% to each other. Considering the lack of a precise standard method for mechanical testing of strands, such close agreement must be regarded as remarkable.

For informational purpose, the testing procedure used at Lehigh University is described in detail here. The first several attempts did not yield satisfactory results, as the stress-strain relationship was distinctly meandering and not smooth. It was also noticed that a twisting of the specimen developed within the gage length during the testing. While the cause for the twisting was not understood, its occurrence was believed to be not desirable. Consequently, a modified procedure was used to eliminate this twisting. In the subsequent tests the strand specimens were preloaded first to a total force of 20,000 lbs. (approximately 130 ksi or nearly one-half the specified tensile strength. The load was then reduced to 4000 lbs. (approximately 26 ksi) and the mechanical extensometer was attached. By virtue of the preloading, a firm grip in the machine heads was assumed, and twisting was eliminated during subsequent loading. Starting at the 4000-lb. load, the elongation of the specimen was recorded at 2000-lb. load intervals until failure. The stress-strain relationship obtained was smooth and initially linear as shown in Fig. 15. As the initial strain reading was recorded at a load of 4000 lbs., the actual strain at this initial load was not known, and was determined indirectly. The linear portion of the plotted curve was extended toward the left to intersect the strain axis (at approximately -0.0009 in./in. strain). This point of zero stress was treated as origin in all subsequence reference to the curve.

5.2 Strand Force Before Transfer

Variations of strand force during the fabrication time, as determined from load cell readings, are shown in Figs. 5 and 6, for the two last fabrication runs. Also shown are the concrete temperatures. It is evident that the strand forces correlated very well with the temperature change. Several observations can be made:

1. The initial stretching stress in the strands was lower than desired, particularly in the draped strands. While the desired initial tension was 28.9 kips per strand, the measured force for the second run (stress-relieved strands) was 28.6 kips in straight strands and 27.8 kips in draped strands. For the third run, using stabilized strands the measured initial tension was 28.9 kips and 28.4 kips, respectively. As the load cells were placed at the dead end of the prestressing end, the under-stress noted above was attributed to frictional resistance at the deflecting devices and bulkheads along the length of the bed. The better agreement for the third run reflected added experience and care in adjusting the deflecting devices to relieve friction.

2. The steel tension decreased significantly during the curing period under the elevated temperature. However, this loss was almost completely recovered after the curing was stopped and the prestressing bed cooled off. Figure 5 shows that when the temperature rose from 50° F (room temperature) to 130° F, the stress-relieved strands lost 1.5 to 1.7 k. of tensile force. These losses correspond to stress changes of

9.1 to 11.0 ksi, or elastic strains of 0.00034 to 0.00038. Using a thermal expansion coefficient of 0.0000065, these strains correspond to a temperature rise of approximately 50° F. This lower temperature rise is interpreted to mean that the strands were sufficiently shielded by concrete. Later, after curing when the temperature dropped back to 50° F, the average measured strand forces for draped and straight tendons both increased by 1.1 kips. The permanent loss of 0.4 to 0.6 kips was very much in line with the anticipated relaxation loss for this initial period of 2.3 days.

Similarly, for the stabilized strands, the loss of total tension during the rise of temperature from 50° to 130° F was 1.7 k, or 11.0 ksi, corresponding to a thermal expansion for 60° F. After the curing, the strand stress increased by 1.5 k or 9.7 ksi. The net loss was 0.2 k over a 45 hour period, again correlating well with the expected relaxation loss during this period of time.

A question has been raised concerning the significance of the test method used for the measurement of strand forces after curing. As all load cells were placed external to the concrete members, it was feared that the measured forces may represent only the conditions in the exposed portions of the strands, and that the embedded strands may not experience the tension recovery after curing exhibited by the load cell readings. It was noted that Eq. (4-4) yielded E_{ci} values considerably lower than those estimated from standard cylinder tests (3480 ksi vs. 3820 ksi for the third run). At first, these disagreements were suspected to support the aforementioned skepticism. A more careful examination, however, relieved this concern. In Figs. 7 and 8, two after-transfer steel stress values are shown, both estimated from the same before-transfer steel stress value. The "predicted" value was based on theoretical elastic analysis using the cylinder modulus, and the "calculated" value was based on Eq. (4-1), which was compatible with Eq. (4-4). In both figures these two f_{so} values agreed within 1% (161.5 ksi vs. 159.8 ksi in one case and 165.9 ksi vs. 164.9 ksi in the other). Furthermore, in both cases the predicted f_{so} was higher than the calculated value. If the before transfer steel stress was actually lower than the value indicated by the load cells, both estimates would be lower. However, the "calculated" value would be lowered more, and the discrepancy would be increased. In other words, the suspected lack of thermal recovery of tendon stress before transfer would have an effect on the estimation of f_{so} contrary to the experimental results. Pending additional deliberate investigation of this question, the experimental evidence appears to indicate that the load cell readings of before transfer steel stress are valid.

5.3 Transfer Length

In Section 4.4 it has been pointed out that after transfer, the prestress in the short prestressed specimens was nearly constant in the middle 20 inches, leaving transfer lengths of approximately 32 inches length at each end. From Figs. 7 and 8 the development of steel stress is seen to be rapid and nearly linear for a distance of approximately

20 in. from each end up to approximately 120 ksi, beyond which the development is more gradual to a maximum stress of about 160 ksi. Within the middle 20 in., the stress remains nearly constant (approximately 160 ksi).

The ACI Building Code (Ref. 9) suggests that the transfer length of the effective prestress can be estimated by the following formula

$$\ell_t = \frac{f_{se}}{3} d_b \tag{5-1}$$

where

ℓ_t = the transfer length, in.

f_{se} = effective stress in prestressing steel, ksi

d_b = nominal diameter of bar, wire, or prestressing strand, in.

This relationship is based on extensive test data and corresponds to an average bond stress of 400 psi, which is attained by virtue of the lateral expansion of pretensioned strands at transfer. For the observed effective steel stress of 160 ksi, upon transfer, Eq. (5-1) yields

$$\ell_t = \frac{1}{3} (160) \; (0.5) = 26.7 \text{ in.}$$

This transfer length compares quite well with the observed value of approximately 32 inches. Allowing for inaccuracies of observation, a length of 30 inches is recommended. No appreciable difference was observed between the two types of strands.

It should be pointed out that the recommended distance of 30 inches accounts for the development of the effective prestress only. Additional distance will be needed for the strand stress to develop further to accommodate the ultimate bending capacity of the section. In the current AASHTO Specifications[17], as well as the ACI Building Code[18], the development length requirement is

$$\ell_d = (f_{pu} - \frac{2}{3} f_{se})\, d_b$$

where

f_{pu} = estimated stress in prestressing steel under the ultimate bending moment, in ksi

ℓ_d = total development length, in inches

Here ℓ_d includes the transfer length ℓ_t and the additional length for further development of steel stress. Comparing this expression for ℓ_d with Eq. (5-1) for ℓ_t, it is seen that the development of steel stress outside of the transfer length is significantly slower than inside that length.

5.4 Concrete Strains at Transfer

Figure 9 shows the measured strain in the shrinkage and short prestressed specimens immediately after transfer. Figure 10 is a similar plot showing the strains in the beam members. In all cases, the essentially linear strain distribution is evident.

It is seen that in no case does the strand type (whether stress-relieved or stabilized) make any difference in the strain distribution. The strain distribution in all shrinkage specimens showed a slight negative gradient, varying from 1 to 2 microstrains per inch, corresponding to a positive bending moment of 180 to 360 kip-in. The source of this bending is attributed to the friction at the bottom and the restraining effect of strands.

It should be noted that the elastic transfer strain is represented by the difference of the measured "after transfer" strain in the prestressed specimen from that in the companion shrinkage specimens. In Figs. 9 and 10 are also shown a line representing calculated total transfer strain in various members. It is seen that they do not differ severely from the measured lines. A significant scatter of strain lines is seen in Fig. 10 indicating differing prestress forces and eccentricities. This is believed to be the result of frictional resistance of the hold-up and hold-down devices and the bulkheads. No relationship between the prestress strain line and the fabrication position of the specimen on the prestressing bed could be ascertained.

5.5 Long-Term Concrete Strains

5.5.1 Strain Distribution Within Specimen

Figures 11 and 12 show the distribution of concrete strain over the depth of two beam members. Plottings for all beam members as well as the prestressed short specimens show similar patterns. The nearly linear strain distribution is easily recognized, also the gradual increase of strain with time. Corresponding to the construction of the deck structure, representing a significant positive bending moment on the member, the strain gradient is seen to decrease as expected. One puzzling phenomenon was observed in all beam specimens. After the transportation of the members, the concrete fiber strains decreased by an amount of from 200 to 300 microinches per inch. As seen from Figs. 11 and 12, this decrease was nearly uniform over the entire depth reflecting an elongation rather than bending. The companion PennDOT research project 71-8 reported no change of camber during transportation, consistent with the finding here of no curvature change. Attempts to interprete this elongation of member during transportation were not successful. This lengthening effect was not observed on the shrinkage or short prestressed specimens. It is suspected that this lengthening may be an illusion, reflecting disturbance to the gage points, rather than real.

5.5.2 Strain versus Time

In all specimens, the concrete strain showed a rapid increase during the initial period of from 10 to 20 days. Beyond this period, the growth of concrete strain became much slower. Plotted against the logarithm of time, the variation of concrete strain appears to be nearly linear, see Figs. 13, 14 and 16.

Figure 13 shows the average strain in each of the four shrinkage specimens. The general tendency for the shrinkage strain to grow with time is easily detectable. Keeping in mind the age differential between the two fabrications runs (approximately 5 days), the behavior of all specimens are remarkably similar until approximately 40 days, when specimens S1, S2 and S3 were moved indoors. From this time on the strain in these three specimens show a rather steady increase with time, while specimen S4 continued to respond to the variation of environment. Quantitatively, the average strain in specimens S1, S2 and S3 reached approximately 200 μ in./in. within three days, fluctuated between 150 and 300 microstrains until 40 days, and afterward increased steadily to about

350 microstrains at 250 days. It is important to point out the logarithmic time scale used in this graph. When the measurements were plotted to a straight time scale, the upward trend was extremely difficult to detect, on account of the expanded time intervals. But there is no doubt of this growth with the aid of the semilogarithmic graph.

Figure 14 shows the long-term concrete strain at the c.g.s. level of the short prestressed specimens. The near linear relationship between strain and logarithm of time, as predicted by the previous research project (PennDOT 66-17, see Ref. 15), is clearly illustrated. The elastic strain in this fiber at transfer time varied for the several specimens, between 800 to 900 microinches per inch. The long-term strain was approximately 400 microstrains in 10 days, 750 units in 100 days, and between 800 and 1100 units in one year. Thus, the concrete strain has nearly doubled in one year's time.

A similar nearly linear growth of total long-term strain is depicted in Fig. 16, where the strains at the c.g.s. line in the beam specimens are plotted. Here, the elastic strain at transfer was approximately 720 microinches per inch, and the long-term strain was 200 within two days, 700 at 100 days, and approximately 650 at one year. Thus, the concrete strain has also nearly doubled in one year.

5.5.3 <u>Indoors versus Outdoors</u>

The effect of varying environment on the concrete strain behavior of specimens can be observed by comparing specimens S3 with S4, P2 with P1, and P3 and P4. In each pair, the first specimen was kept indoors under a rather constant condition, while the second member was subjected to the fluctuation of outdoor environment.

Specimens S3 and S4 behaved almost identically at the beginning when both were stored outdoors in the fabricator's yard. After specimen S3 had been moved indoors on March 14, 1972 (36 days), the behavior of the two specimens were considerably different. It is seen that S3 exhibited a rather steady increase of strain while specimen S4 exhibited much more wide fluctuation, obviously responding to the relation humidity in the environment. While S3 showed a total strain of 220 μ in./in. in 40 days, 300 μ in./in. in 100 days, 320 μ in./in. in 160 days, and 340 μ in./in. in 240 days, the corresponding strains for S4 were 160, 320, 220 and 340 μ in./in., respectively. However, no definite trend as to the final magnitude of the shrinkage strains in this specimen can be established at this time.

The comparison between the indoor and outdoor prestressed specimens yielded more definite results. The indoor specimens showed significantly larger long-term strains. At the end of one year the strains were 910 and 1100 μ in./in. for the two indoor specimens, and 870 and 800 μ in./in. for the respective outdoor specimens. It is also clear that the outdoor specimens showed more severe fluctuating of strain from time to time.

It is remembered that the paired short prestressed specimens were separated at an age of approximately 3 days, while the shrinkage specimens were stored together until they were 36 days old. This difference could probably explain the similarity of the projected final strain values in the shrinkage specimens. Apparently, the total shrinkage strain in a member is more sensitively controlled by the environmental condition during its early stages. In line with this thinking, it could be expected that the in-service bridge beams would experience less shrinkage strain than predicted by the previously developed procedure.

5.5.4 Stress-Relieved Strands versus Stabilized Strands

The effect of strand type was observed by comparing the long-term concrete strains in paired short prestressed specimens P2-P3 and P1-P4, as well as the beam member specimens. Figures 14 and 16 show the average measured long-term concrete strains at the third level of Whittemore gage target points, which are close to the c.g.s. line.

The two indoor short prestressed specimens, P2 and P3, behaved almost identically up to approximately 150 days, after which specimen P3, containing stabilized strands, showed higher strains, indicating higher remaining stresses, or lower prestress losses, as might be expected. In contrast, the two field stored short prestressed specimens, P1 and P4, behaved somewhat differently from very early times. While the strain in both specimens fluctuated in a similar pattern with environmental conditions P1, containing stress-relieved strands, unexpectedly registered higher strains than P4. All beam specimens showed approximately the same concrete strain throughout this period of observation, although as time proceeds, a slight tendency may be detected for beam Nos. 3 and 11, (stress-relieved strands) to register lower concrete strains.

Quantitatively, it is seen that both P2 and P3 registered average concrete strain at the c.g.s. level of approximately 400 microinches per inch at 7 days, 700 at 50 days, 800 at 100 days, and 830 at 150 days. After 150 days, strain in P3 increased more rapidly. At 350 days, strains from these two specimens were 1080 and 970 microinches per inch, respectively.

Quantitative comparison of strains in the outdoor specimens was less convenient to make because of the interference of fluctuation due to environmental conditions. In Fig. 14 the graphs for specimens P1 and P4 show obviously similar fluctuations, but with a time shift of approximately five days. This is because of the different casting date of these two specimens. The time shift becomes less visible later on account of the logarithmic time scale. In the following comparison, this time shift effect was given consideration. P1 and P4 behaved differently from early times. At ages of approximately 7, 50, 100, 150 and 350 days, the strains in these two specimens were approximately 500 vs. 300, 530 vs. 430, 810 vs. 720, 730 vs. 620, and 870 vs. 800 microinches per inch, respectively. It is interesting to note that these pairings of strain values all have approximately the same difference of 100 microinches per inch.

Figure 16 shows that all beam specimens experienced approximately the same strain growth up to approximately 100 days. At ages of 5, 40 and 90 days, the strains were in ranges of 230 to 370, 400 to 450 and 580 to 650 microinches per inch respectively. After 100 days, specimens 3 and 11 registered slightly lower strains. At 170 and 350 days, the average strains in these two specimens, containing stress relieved strands, were 590 and 610 microinches per inch, respectively, while the corresponding values for beams 4 and 5, containing stabilized strands, were 650 and 650 microinches per inch. Compared with the scatter of data within each group, the inter-group difference must be considered rather insignificant.

The lack of consistent and pronounced differences between the stabilized strands and the stress-relieved strands was not initially expected. After reviewing the member design, however, it became apparent that this should be the case. As pointed out in Section 2.2, the beam members were designed to be inordinately shallow and heavily prestressed. The prestressing parameter β, defined in Section 4.4, is 50.5 for the middle section of the beam, which is near the lower limit of its range for all bridge members (see Ref. 15). In hindsight, it must be concluded that this particular beam design was unfortunately not suitable for the study of the effect of using stabilized strands. For heavily prestressed members, prestress losses are little affected by the relaxation characteristics of the steel. Figure 7 of Ref. 15 shows that at $\beta = 50.5$, relaxation accounts for only about 7% of the total prestress loss. It is therefore not unreasonable that similar behavior would be observed for all specimens, regardless of the strand type. The differences noted between the paired short prestressed specimens were apparently caused by factors other than the characteristics of the strands. In particular, the near constant difference between strains in P1 and P4 tends to suggest that it was caused by some one-time disturbance rather than the continuous influence of innate material properties.

6. CONCLUSION

Based on the observations made in this reported study, the following preliminary conclusions can be deduced.

1. The prestressing force decreases significantly during the curing period, but the loss appears to be completely recovered after the curing was stopped. The net loss before transfer is consistent with what would be expected from strand relaxation.
2. The transfer length of the 1/2" strands may be estimated at 30 inches. No distinction needs to be made in this respect between the stress-relieved and stabilized strands.
3. Indoor specimens suffer higher shrinkage and creep strains than the outdoor counterpart.
4. Concrete strains obey linear distribution over the depth of the member throughout the period of observation.
5. For the geometric design of the specimens used in this study, the type of strand used have little effect on the member behavior. However, as time increases, a tendency can be detected that the use of stabilized strands resulted in lower prestress losses.

7. ACKNOWLEDGMENTS

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TABLE 1

TEST SPECIMENS

Specimens	Strand Type: Stress-Relieved	Strand Type: Stabilized	Storage: Indoors	Storage: Outdoors	Prestress: Yes	Prestress: No	Deck: Cast in Place (Span 2)	Deck: Precast (Span 1)	Position on Bridge (See Fig. 1b)
Main Beams									
3	X			X	X		X		3
4		X		X	X		X		4
5		X		X	X		X		5
9		X		X	X			X	3
10		X		X	X			X	4
11	X			X	X			X	5
Short Prestressed Specimens									
P1	X			X	X				
P2	X		X		X				
P3		X	X		X				
P4		X		X	X				
Shrinkage Specimens									
S1	X		X			X			
S2	X		X			X			
S3		X	X			X			
S4		X		X		X			

TABLE 2

FABRICATIONS SCHEDULES

Fabrication Run	No. 2			No. 3		
Strand Type	Stress-Relieved			Stabilized		
Specimens Cast	3, 11 P1, P2, S1, S2			4, 5, 9, 10 P3, P4, S3, S4		
Action	Date	Time	Hours from Start	Date	Time	Hours from Start
All strands threaded	2-1-72	9:30		2-5-72		
Initial tensioning began		15:30	0	2-7-72	9:20	0
completed		17:30	2		10:15	0.9
Final tensioning began		20:00	4.5		11:00	1.7
completed		20:40	5.2		13:50	4.5
Concreting began	2-2-72	12:00	20.5	2-8-72	8:00	22.7
completed		20:40	29.2		16:00	30.7
Curing began		22:55	31.4		17:30	32.2
completed	2-3-72	19:00	51.5	2-9-72	18:15	56.9
Nine top strands cut		21:30	54.0		20:20	59.0
Group detensioning began		23:40	56.2		22:30	61.2
completed	2-4-72	0:30	57.0		23.30	62.2

TABLE 3

PROPERTIES OF CONCRETE

Date or Age		Fabrication Run #2		Fabrication Run #3	
		f'_c (ksi)	E_c (10^6 psi)	f'_c (ksi)	E_c (10^6 psi)
Transfer	Design Value	5.0	--	5.0	--
	Test(1)	5.11	--	4.96	--
	Test(2)	6.38	(4.27)(3)	5.75	(4.06)(3)
28 day	Design Value	5.5	--	5.5	--
	Test	7.40	4.60	7.74	4.79
58 day		8.25	4.83	8.14	4.91
July 25, 1972 (Casting of Deck)		7.86	5.10	7.39	4.88
October 13, 1972 (First Traffic Load)		8.43	5.21	7.80	5.01

(1) At plant, approximately 5 hours before transfer.

(2) At Lehigh University, 10 and 12 hours, respectively after transfer.

(3) Estimated, compressometer malfunctioning during the test.

Note: Each tabulated test value represents the average of two to three standard cylinder tests.

TABLE 4

PROPERTIES OF PRESTRESSING STRANDS

		Stabilized Strand	Stress Relieved Strand
Diameter of single wire (outer)(in.)		0.1665	0.1671
Diameter of central wire (in.)		0.173	0.1731
Diameter of strand (in.)		0.505	0.5071
Cross section (in.^{2})		0.154	0.155
Lay (in.)		7.0	7.1
f_{py}** (ksi)	L. U.	252.4	248.3
	CF & I	259.0	248.3*
	PennDOT	259.0	249.7
E_s** (ksi)	L. U.	28350.0	28600.0
	CF & I	28750.0	27800.0*
	PennDOT	28050.0	27250.0
f'_{ps}** (ksi)	L. U.	278.0	278.1
	CF & I	279.0	270.0*
	PennDOT	276.0	274.5

*From typical stress-strain curve

**Stresses were calculated based on typical cross section area of 0.153 sq. in.

Note: All tabulated values represent averages of measurements on three specimens from each reel of strands used in the concrete members.

TABLE 5

TYPICAL SCHEDULE FOR CONCRETE STRAIN MEASUREMENTS

(Age of Specimens in Days)

1	28	112	308
3	42	140	364
5	56	168	455
7	70	196	546
14	84	224	637
21	98	252	728

Note: In addition, strain measurements were made before and after transfer of prestress, after erection of beams, and after casting of deck slab.

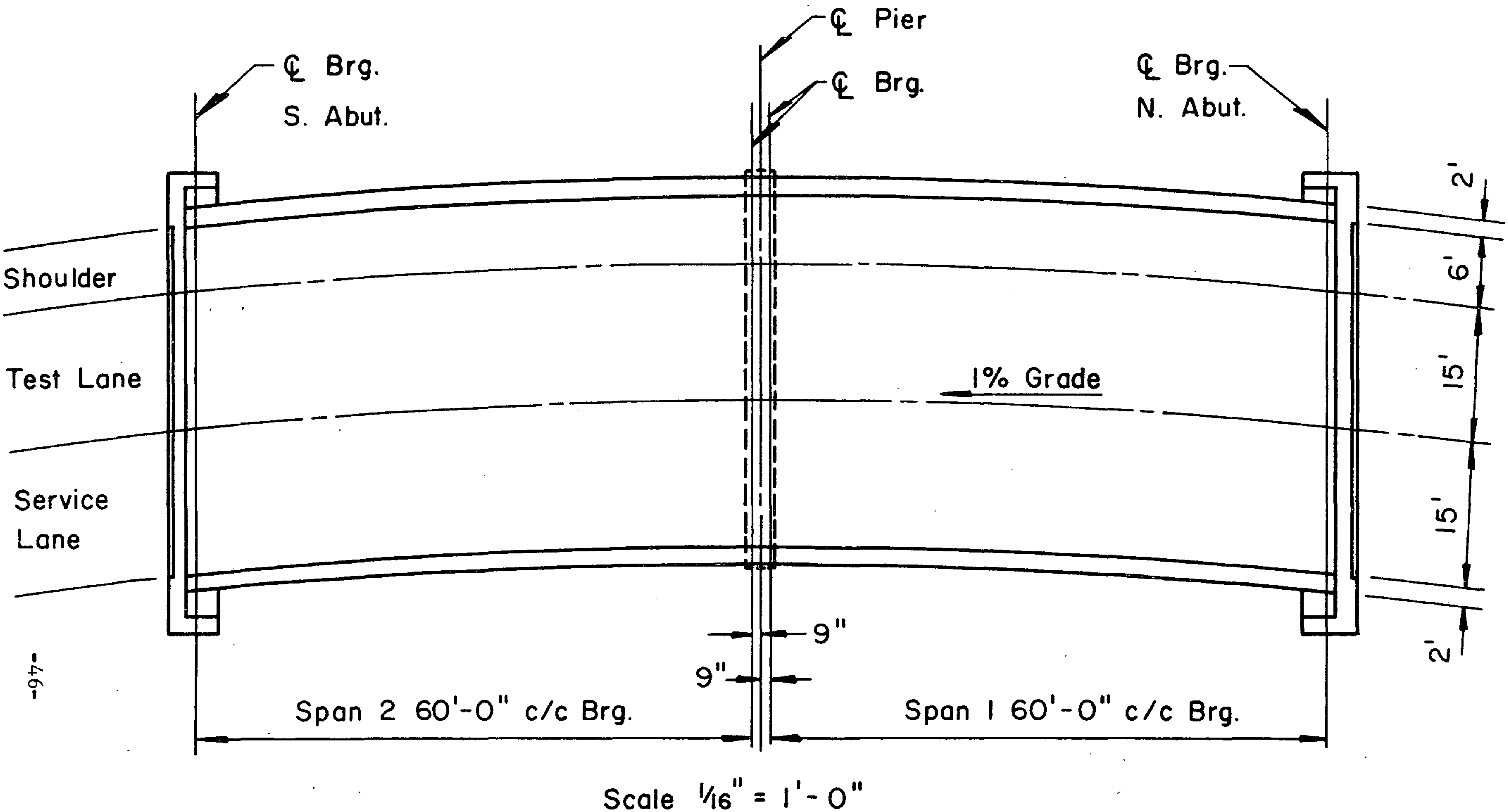
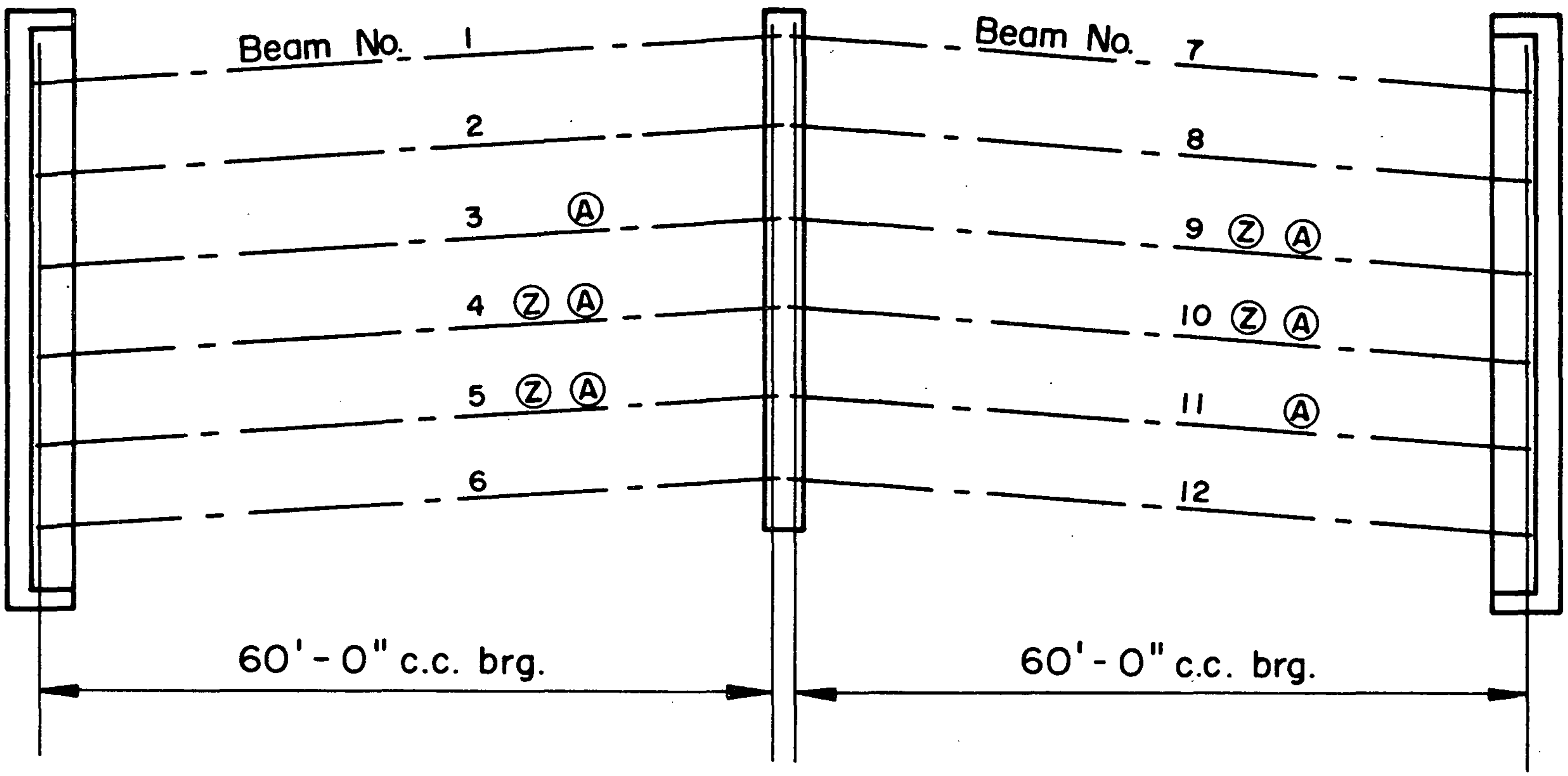


Fig. 1a Plan of Experimental Bridge



Z Stabilized Strands   A Instrumented

Fig. 1b Framing Plane of Main Beams

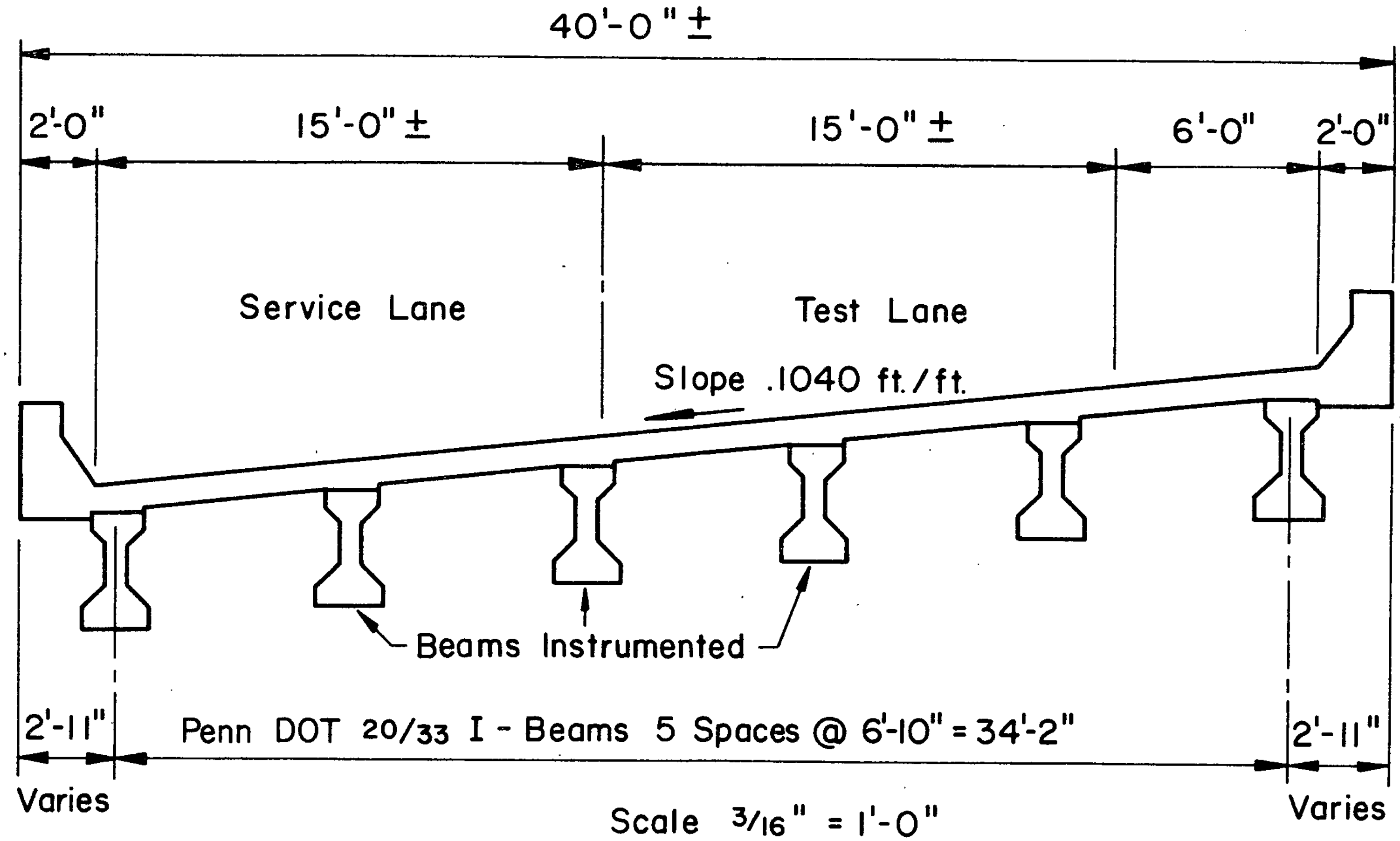


Fig. 2 Typical Cross-Section of Experimental Bridge

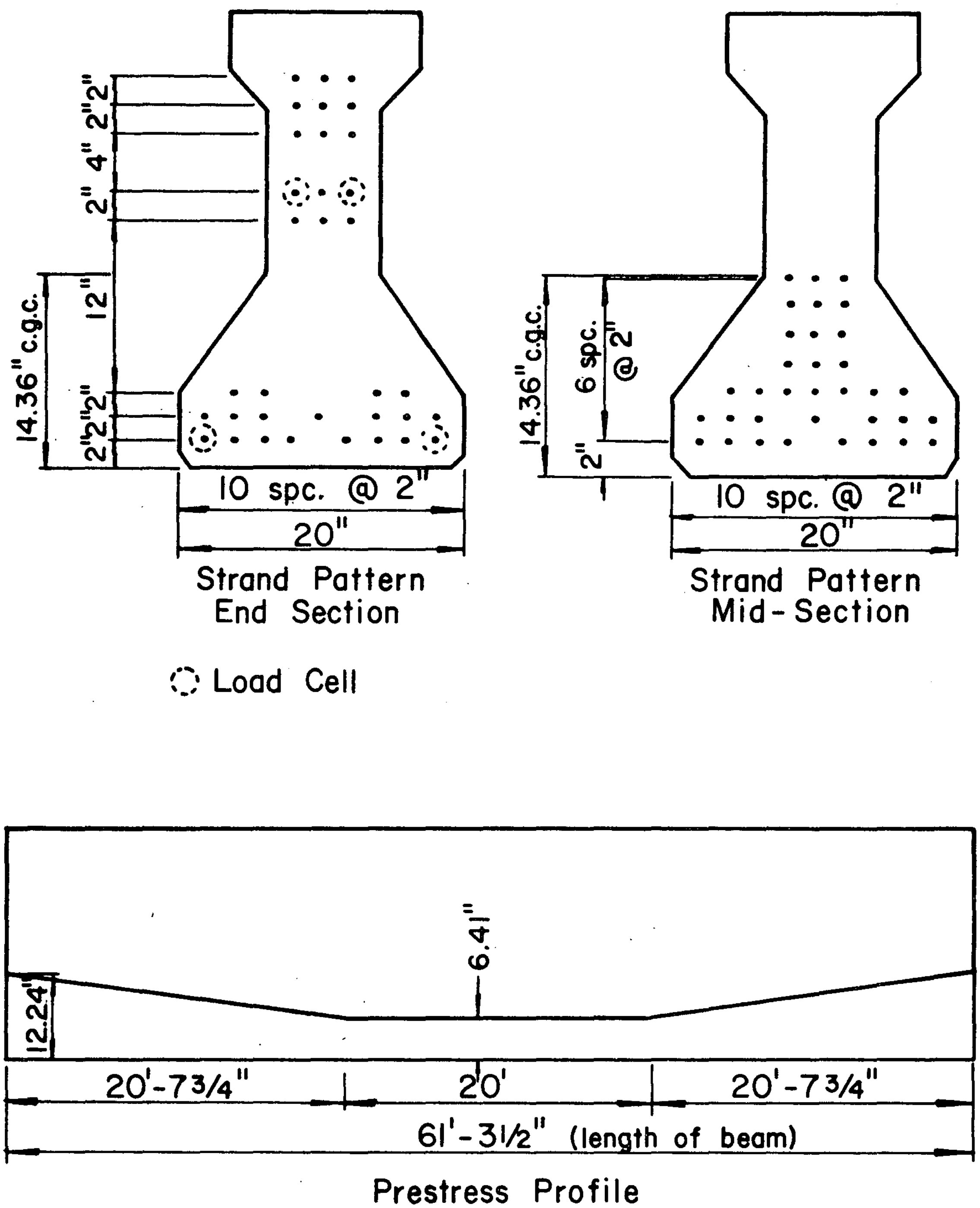
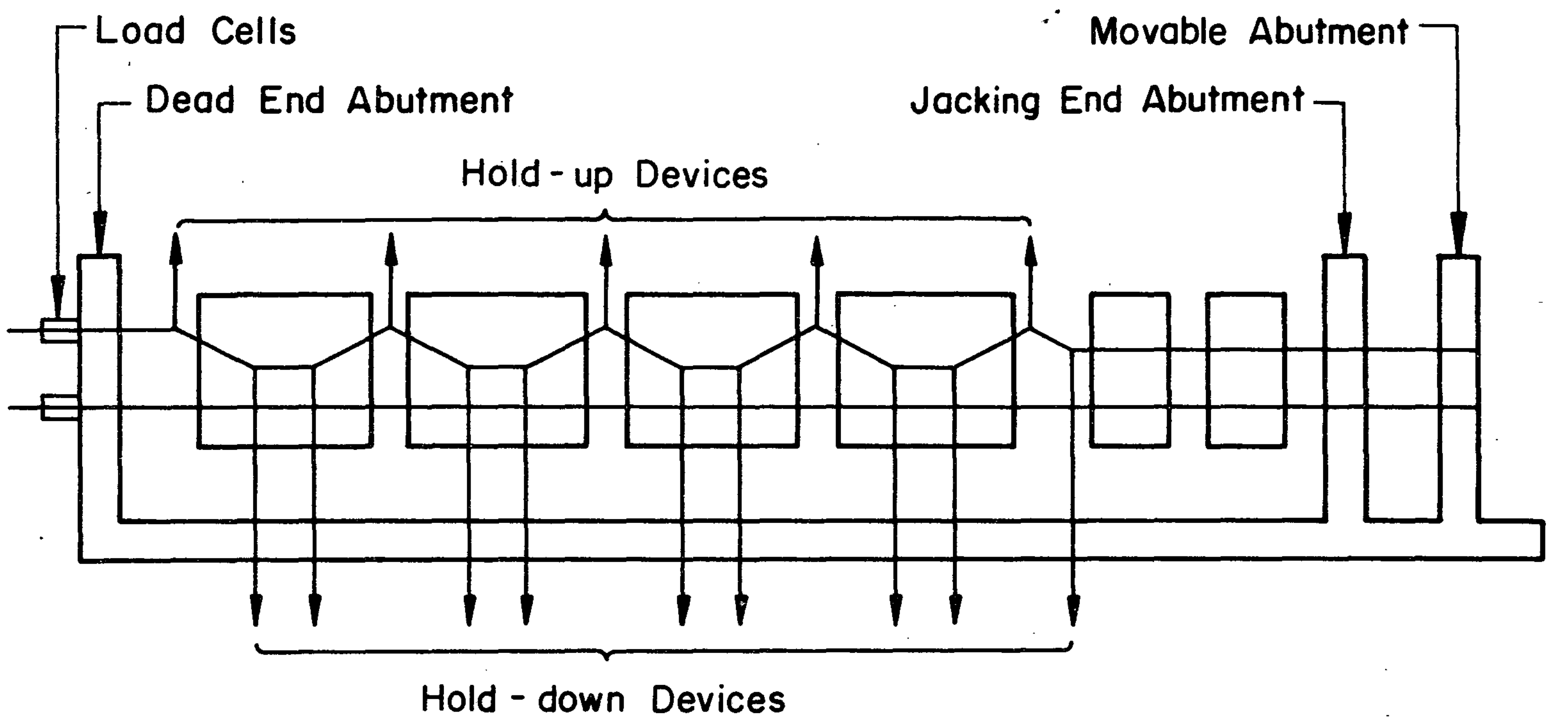


Fig. 3a Typical Cross-Section of Beam and Profile of the Strand Along the Beam

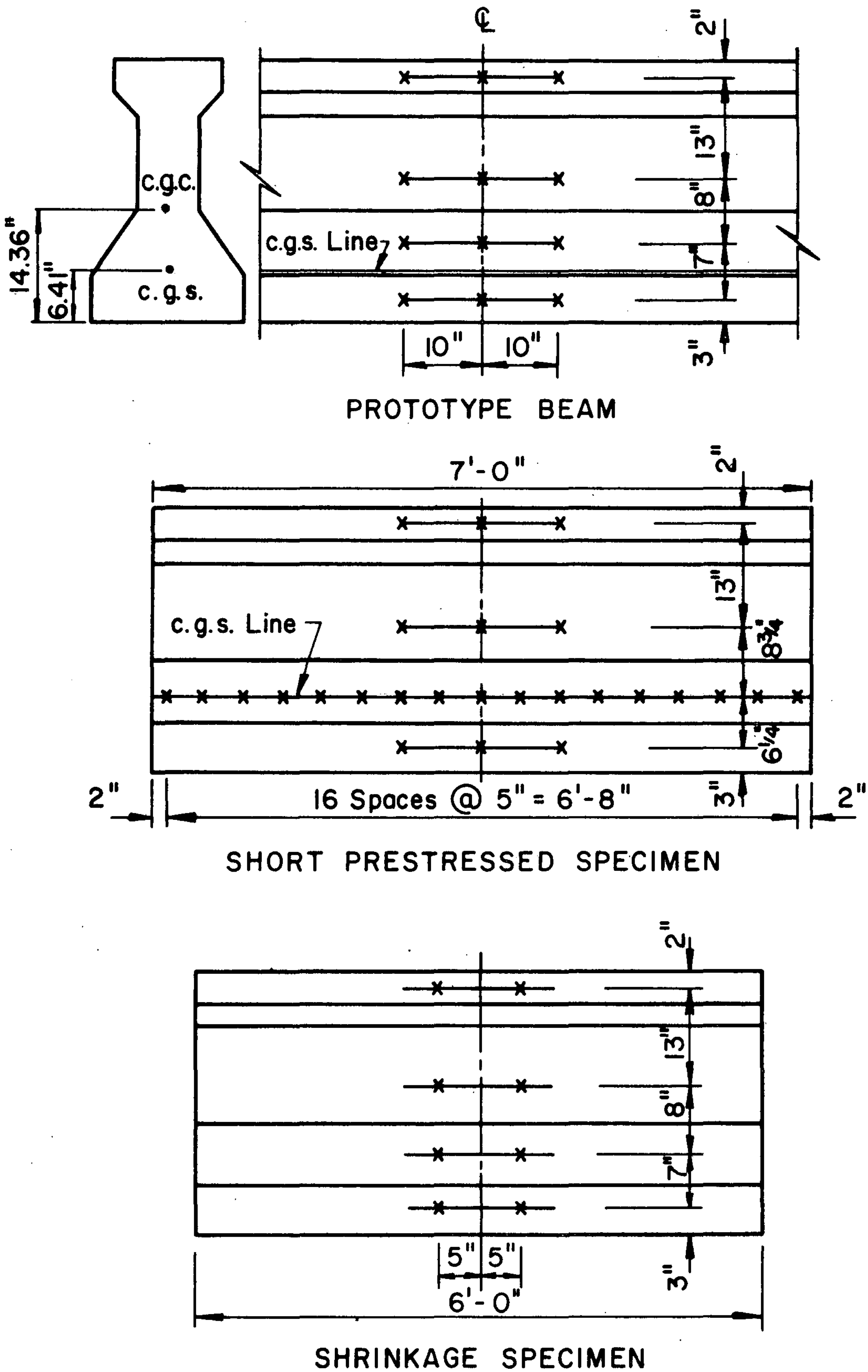


Specimen Numbers

Run *2	3	8	11	7	P1	P2	(S1, S2)*
Run *3	4	5	9	10	P3	P4	(S3, S4)*

* These specimens were cast an adjacent bed.

Fig. 3b Schematic Bed Layout for Fabrication



x Denotes Whittemore Gage Target Points

Fig. 4 Locations of Target Points

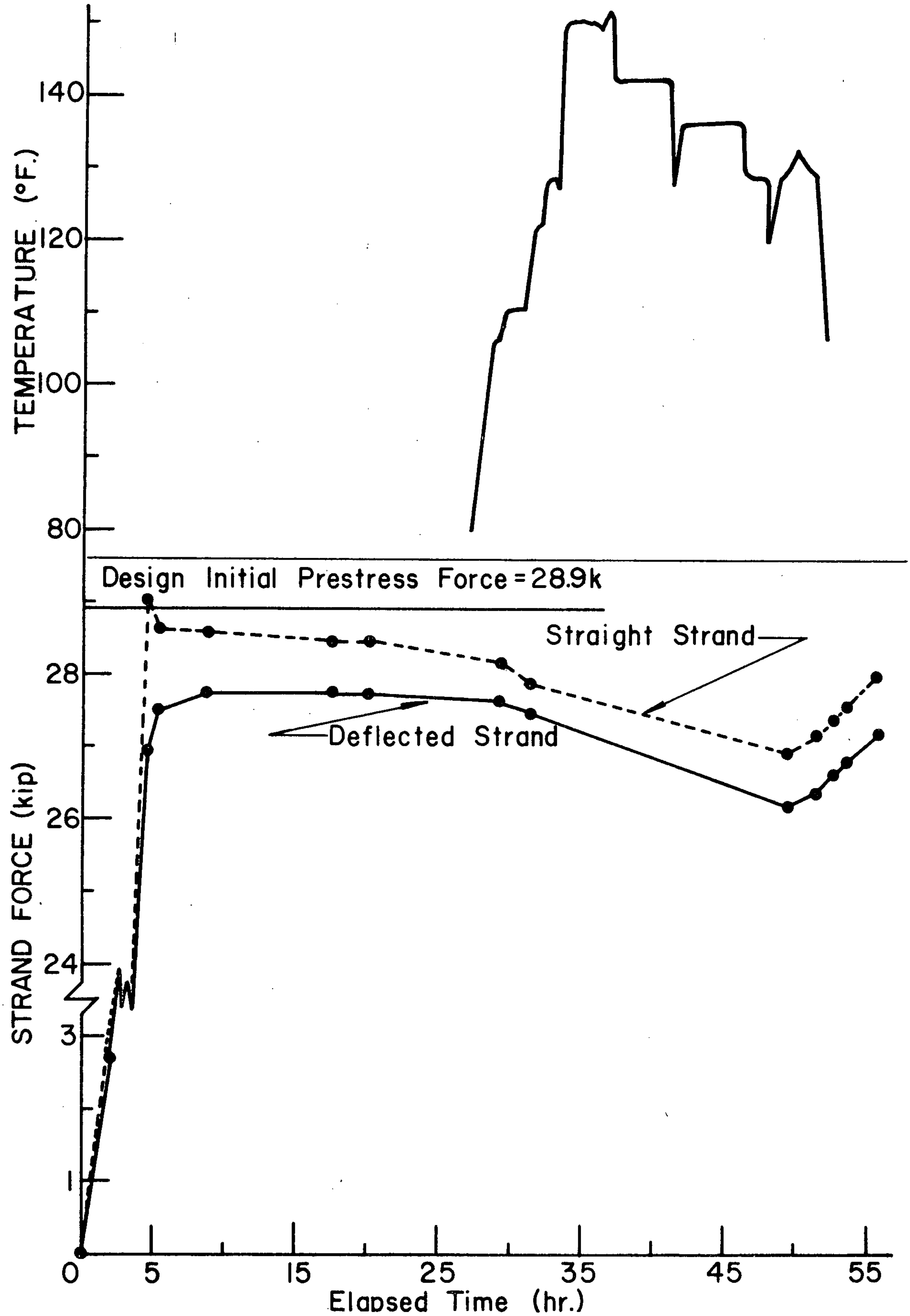


Fig. 5 Strand Force and Curing Temperature before Transfer - Stress-Relieved Strands

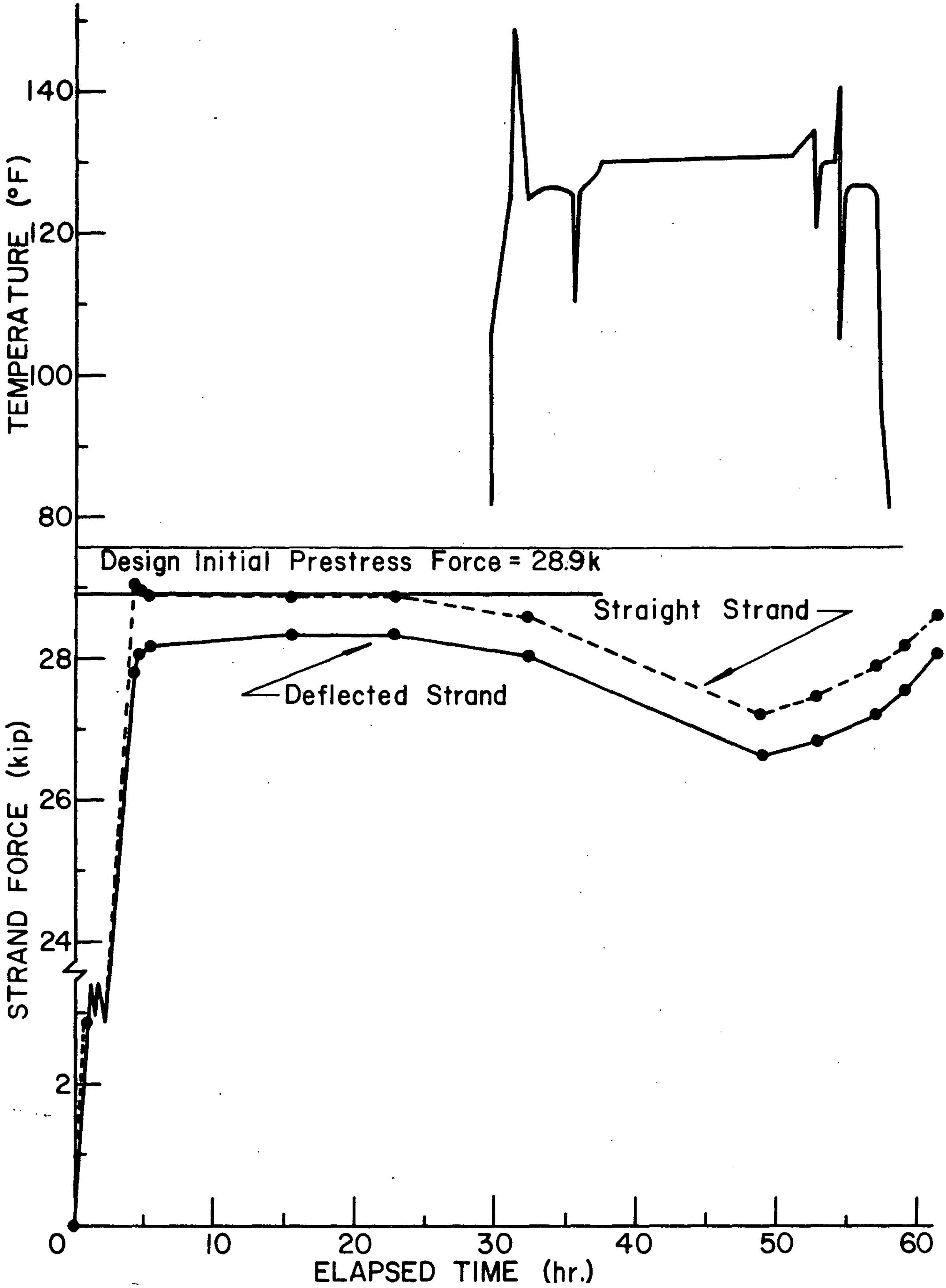


Fig. 6 Strand Force and Curing Temperature before Transfer - Stabilized Strands

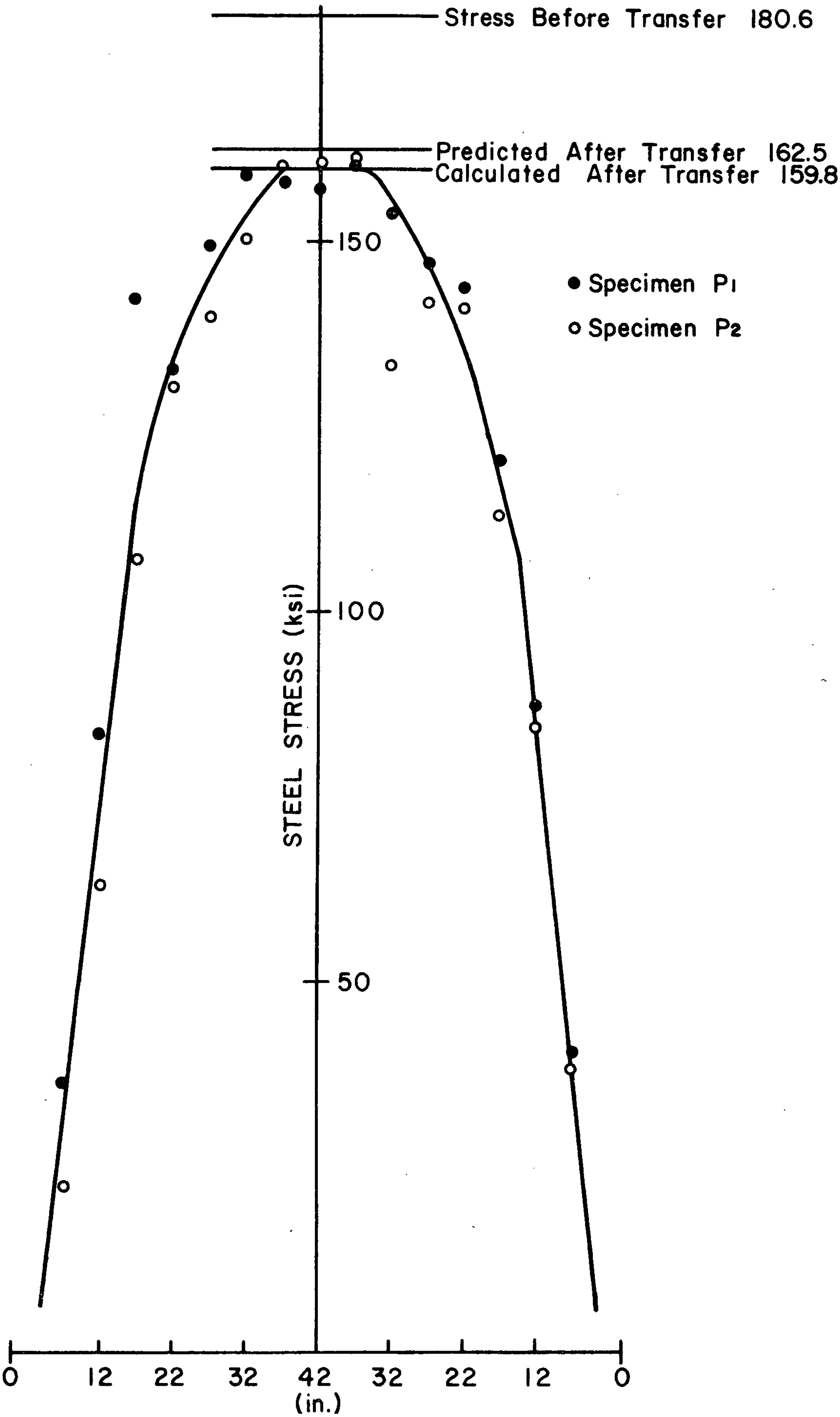


Fig. 7 Development of Prestress - Stress-Relieved Strands

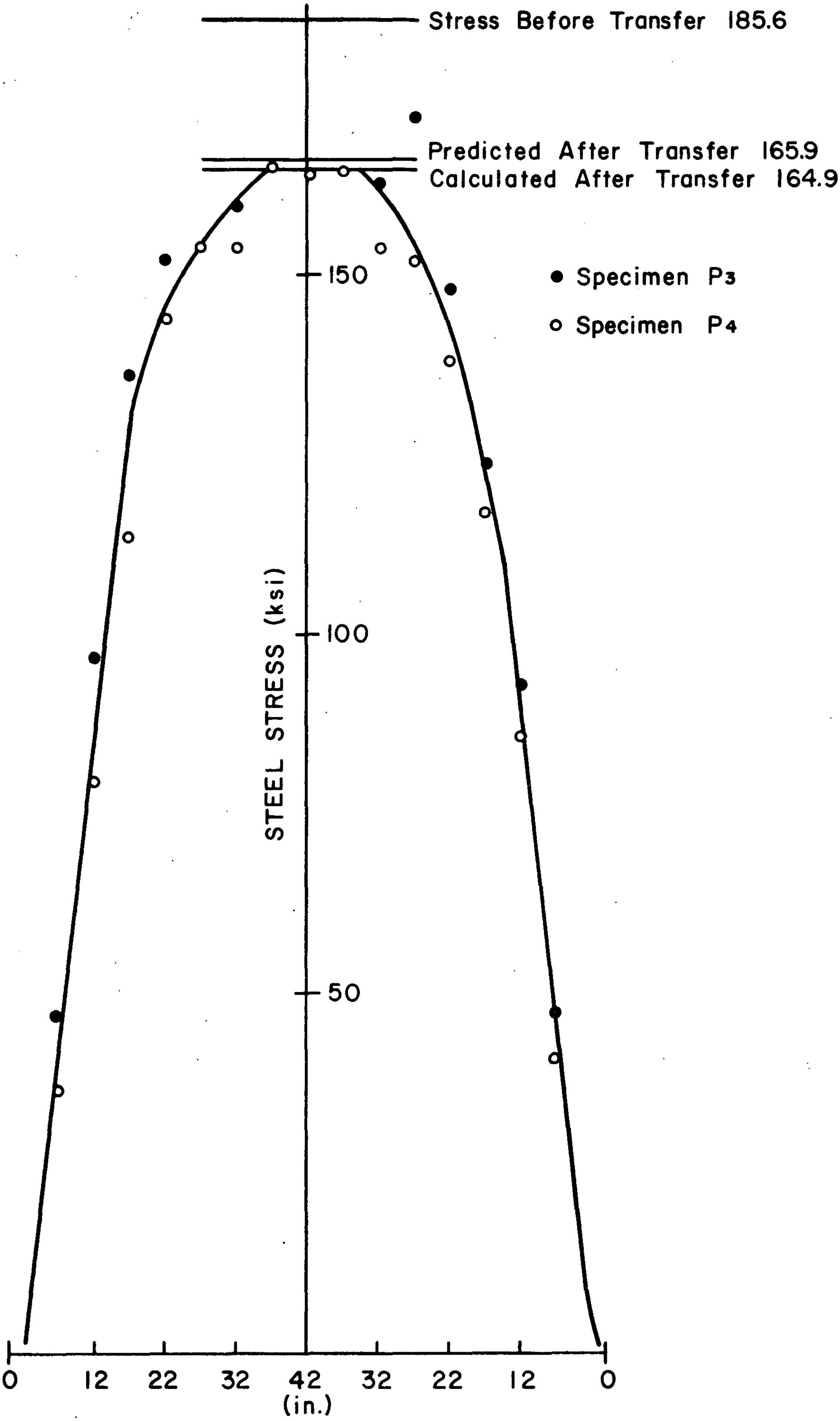


Fig. 8 Development of Prestress - Stabilized Strands

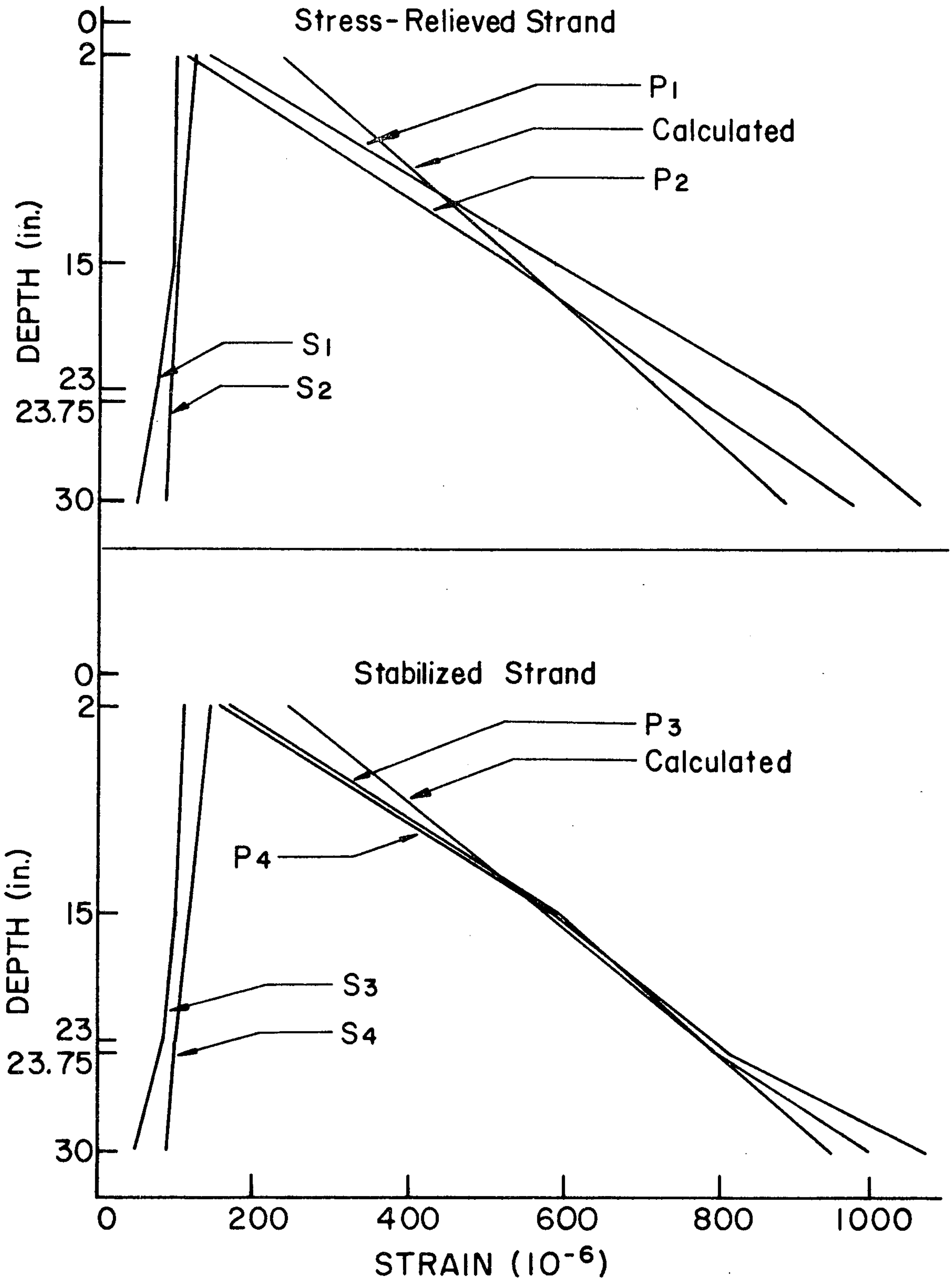


Fig. 9 Concrete Strains at Transfer, Control Specimens

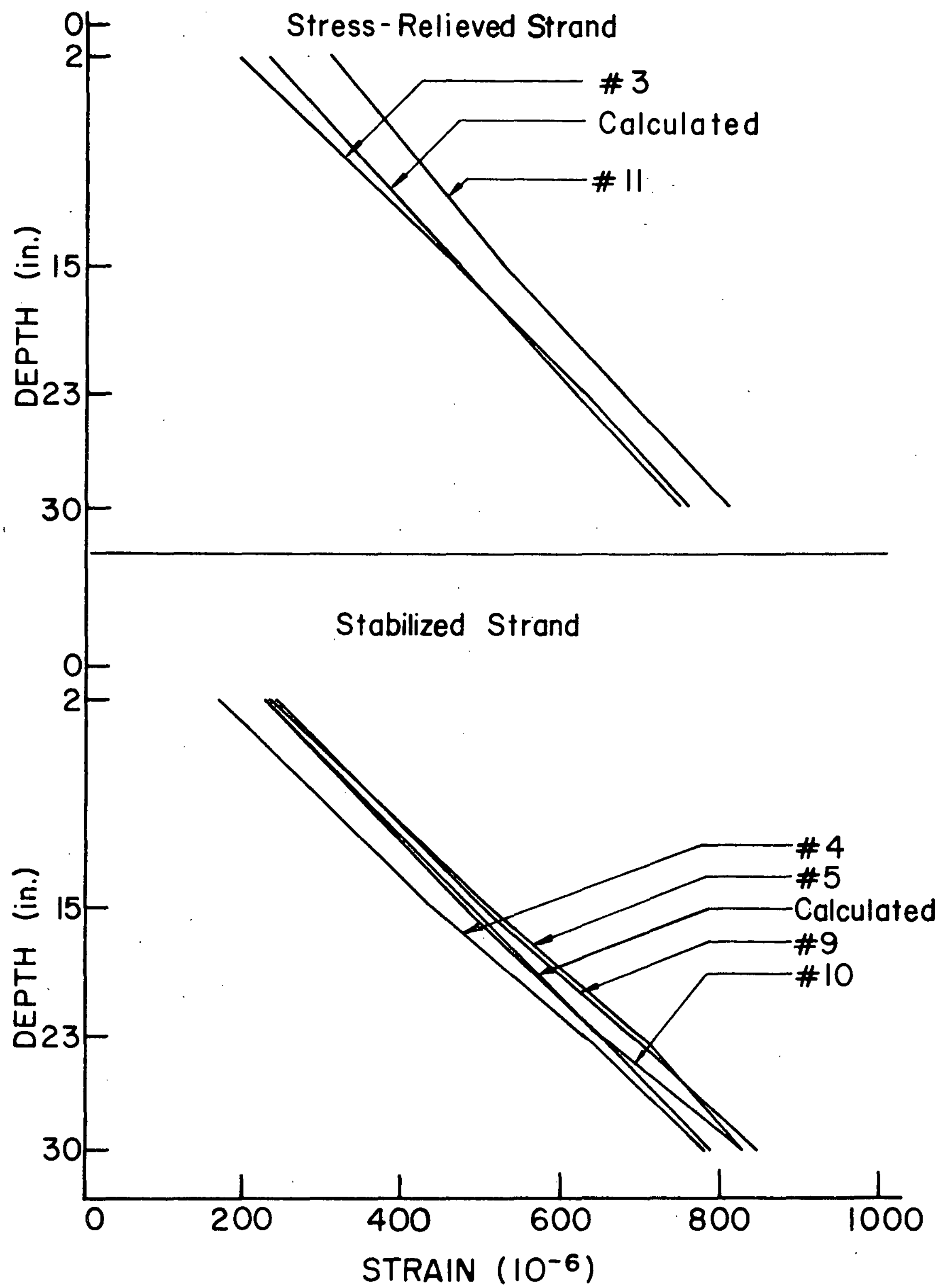


Fig. 10 Concrete Strains at Transfer, Beam Specimens

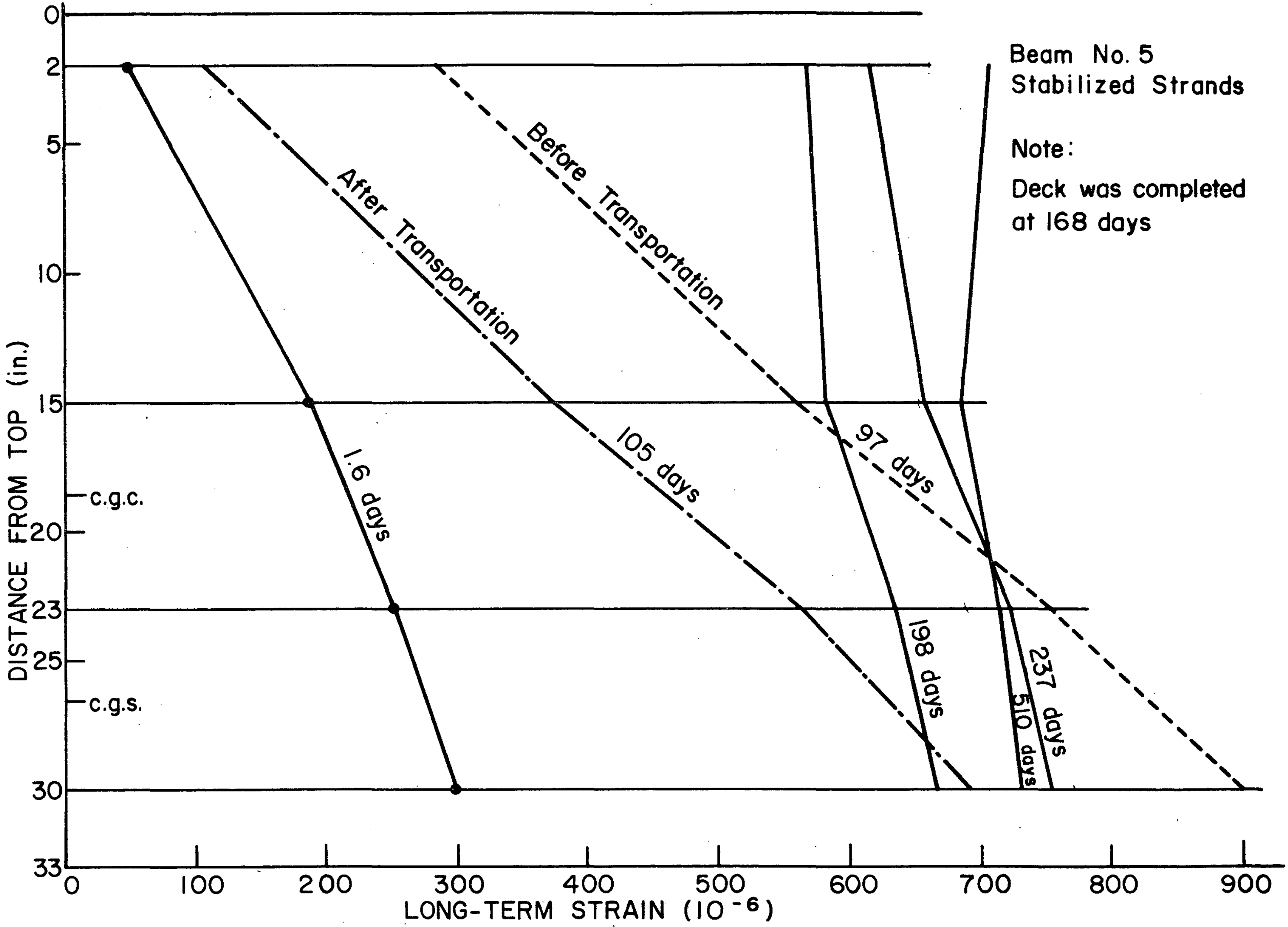


Fig. 11 Distance from Top vs. Concrete Strain Curve for Beam No. 5

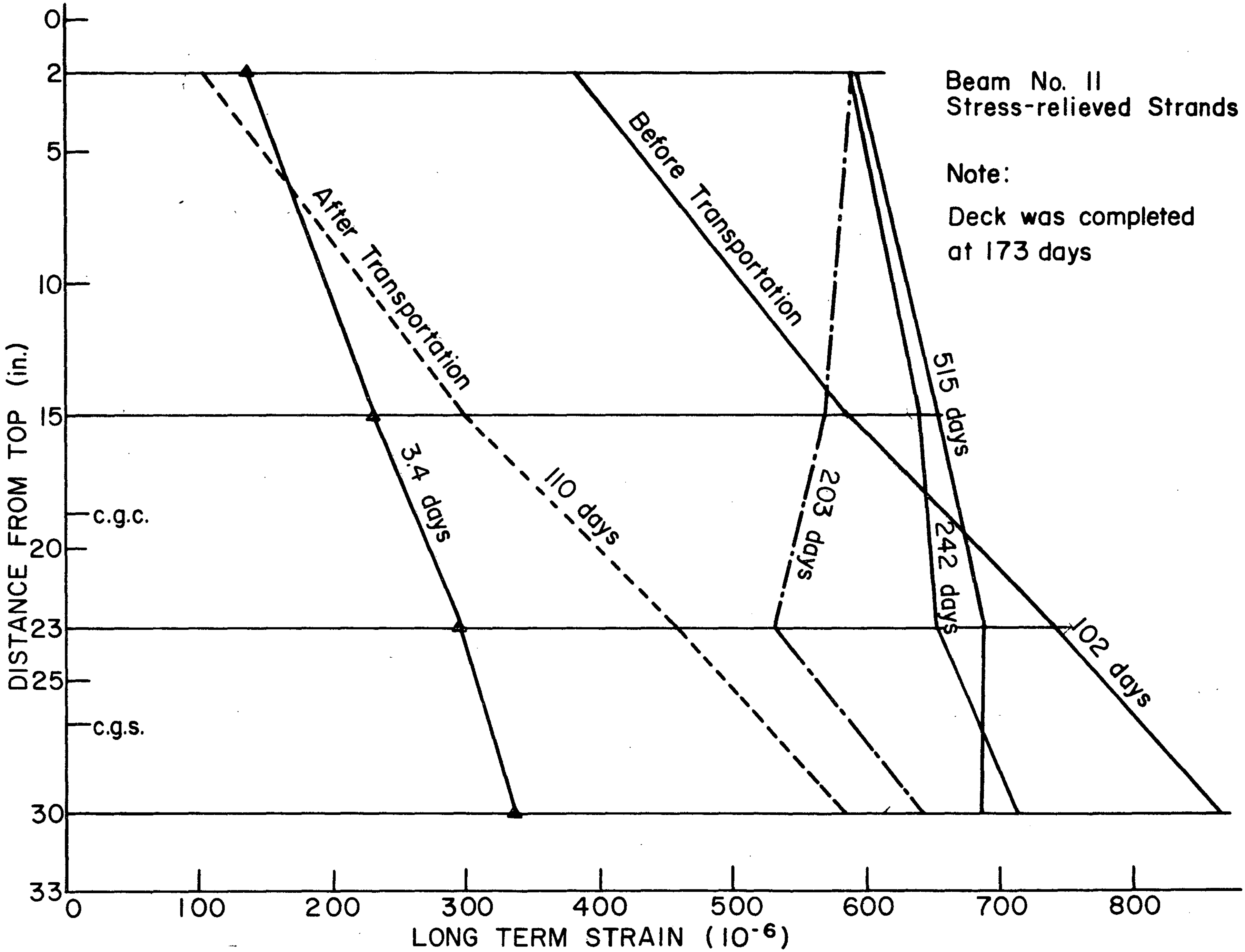


Fig. 12 Distance from Top vs. Concrete Strain Curve for Beam No. 11

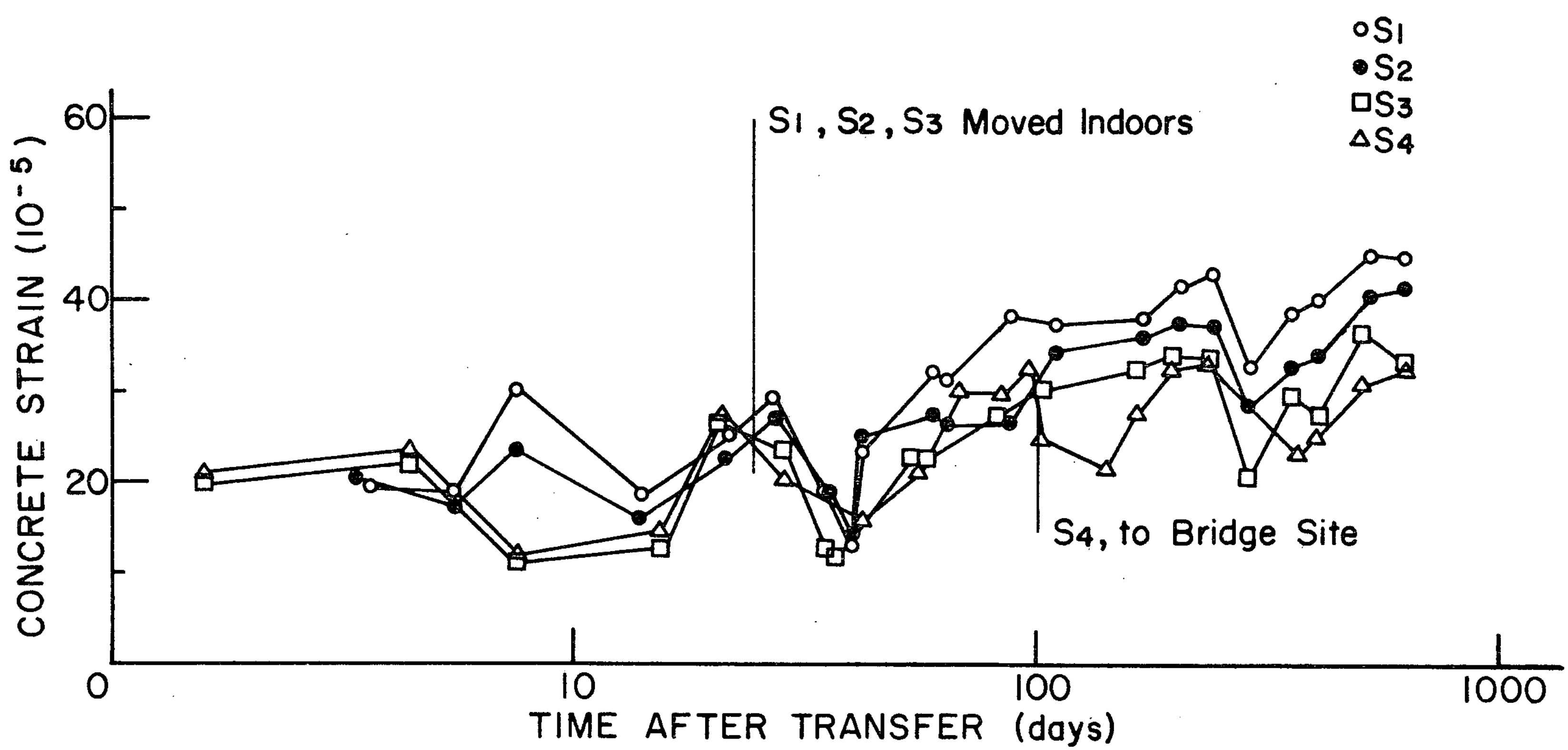


Fig. 13 Average of Concrete Strain vs. Time Curve for Shrinkage Specimens

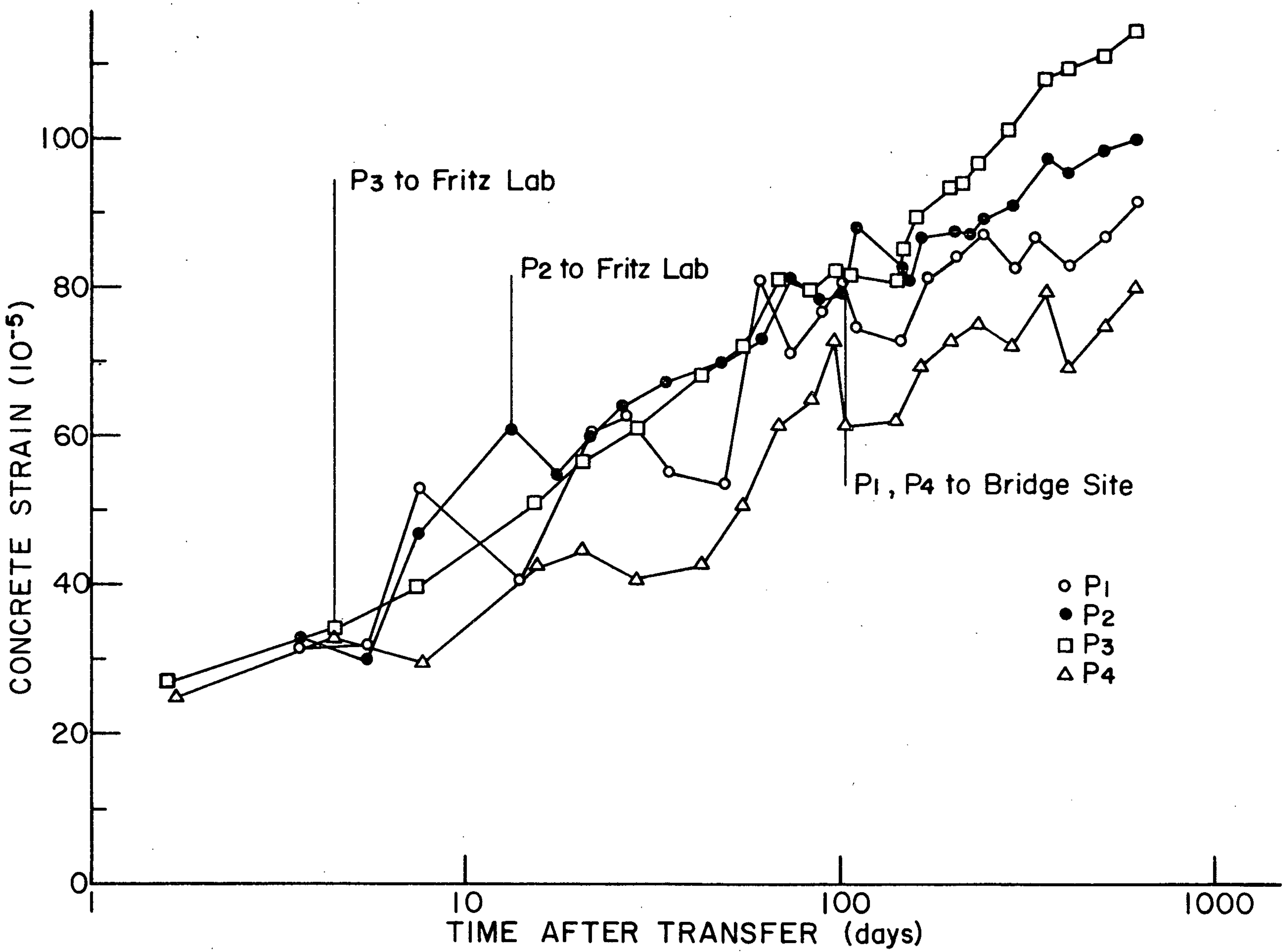


Fig. 14 Average of Concrete Strain vs. Time Curve for Short Prestressed Specimens

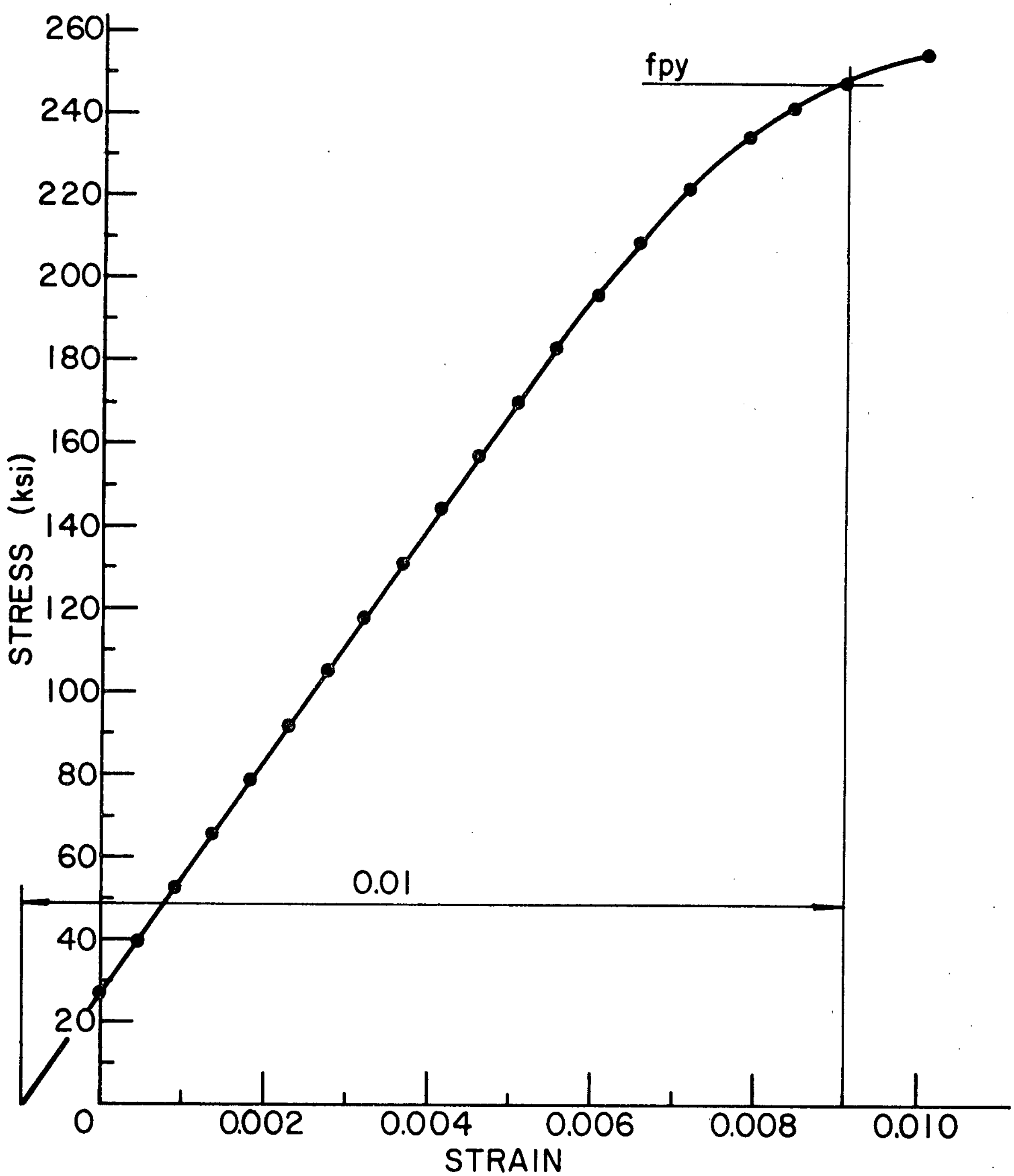


Fig. 15 Stress-Strain Curve for 1/2" Stress-Relieved Strand

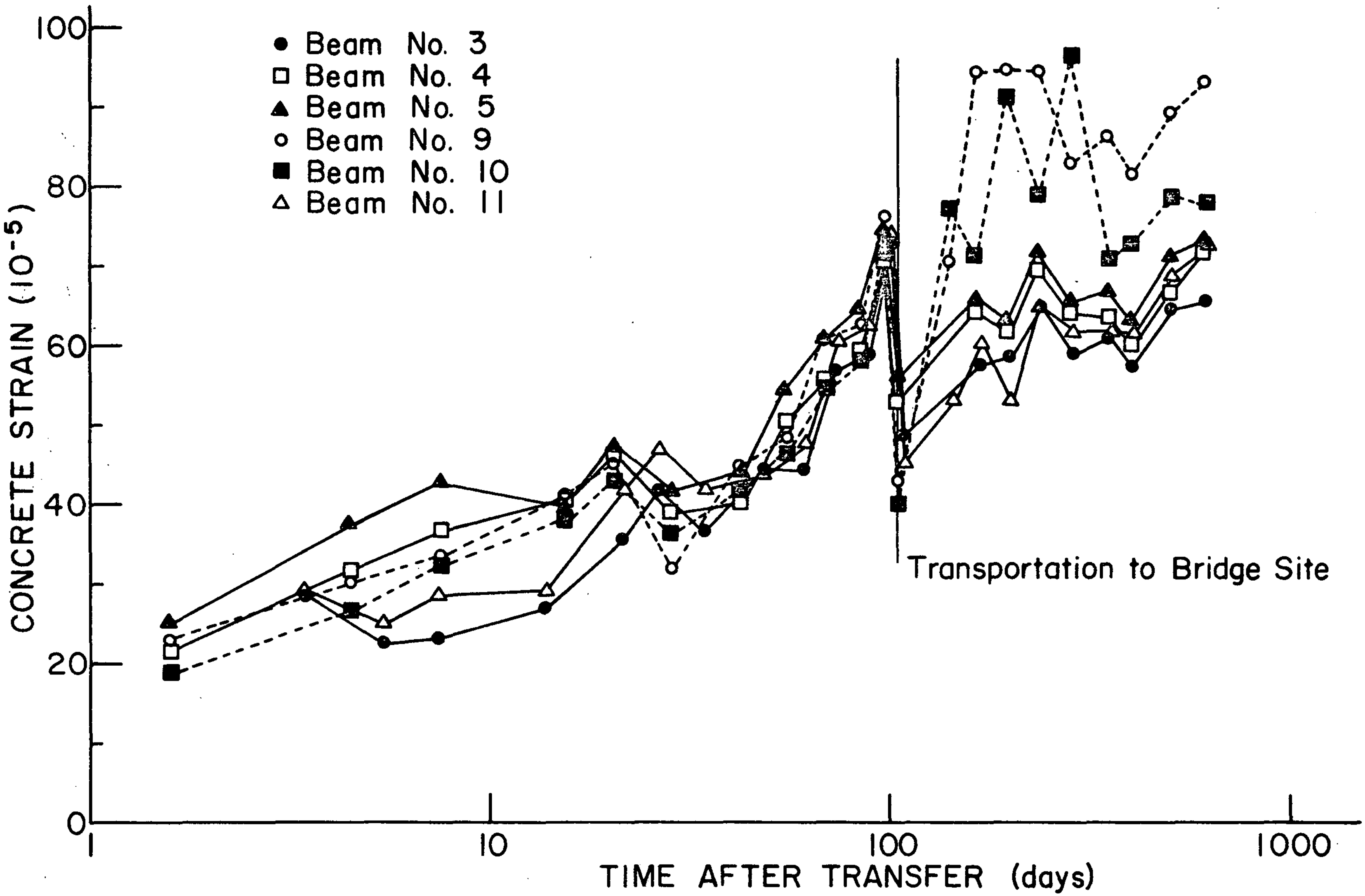


Fig. 16 Average of Concrete Strain vs. Time Curve for Beam Specimens

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